THE STRUCTURAL USE OF PARTICLEBOARD

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STATEMENT OF SOURCES

To the best of my knowledge, the work contained within this thesis is original, except as acknowledged in the text. It does not include any work submitted for the award of another degree or diploma at this or any other university.

[redacted]

Jack Arnold Taylor

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NOTATION

N_j	charactertic joint strength
Q^*	estimated design load effect
Q_k	characteristic fastener strength for limit states design
Q_{BASIC}	fastener strength for working stress design
Q_E	test load
R	structure, component, or joint resistance
R^*	estimated structure, component or joint resistance
$R_{0.05}$	characteristic 5%ile strength
R_{BASIC}	estimated resistance by working stress methods
$\overline{R}$	mean resistance of structure determined by computation or testing
$R_{k,norm}$	normalized characteristic property
S	load effect the structure is designed t resist
$\overline{S}$	mean load effect
S^*	estimated load effect
V_B	variability of as-built structure resistance
V_C	variability in construction
V_D	variability in design
V_M	variability of material properties
V_R	variability of structure, component, or joint resistance
V_S	variability of load effects
V_T	variability of test load
X	geometric property appropriate to failure mode

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f_0'	5%ile characteristic strength property
j_3	duration-of–load factor for deformation
k_1	duration-of-load factor for strength
k_6	temperature/humidity factor
k_{28}	sampling factor
p_F	probability of failure
β	reliability index
γ_0	over-all structure resistance reliability factor
γ_1	structure importance factor
γ_j	joint reliability factor
γ_R	resistance reliability factor
γ_S	load reliability factor
$\gamma_{S,typ}$	typical load reliability related to AS 1170
γ_T	test reliability factor
ϕ	capacity factor as used in AS 1720.1
$\phi_{0,3.0}, \phi_{0,3.5}, \phi_{4.0}$	capacity factors for β =3.0, 3.5, 4.0 respectively as used in AS 1720.1
ϕ_B	capacity factor related to as-built variability

ABSTRACT

The structural use of particleboard is presently restricted because appropriate design data are not included in the Australian timber structures design code, AS 1720-1997, *Timber structures*. The data required needs to be underpinned by evidence that its durability and mechanical properties are suitable for general use and that long-term behaviour under load is at least as predictable as that of timber. It is demonstrated through this study that, when situated in sheltered environments, durable structures incorporating primary particleboard elements are feasible and predictably reliable in the context of normal limit states design methods and common trade practice.

The reliability of timber structures generally is uncertain due to the influence of two unrelated factors. One is that the rheology of timber and reconstituted wood panels is not understood. The other is the limited data supporting some aspects of probabilistic design methods.

With respect to the former, current predictive models are unable to forecast behaviour beyond the period of continuous observation, most of which is limited generally to one or two years and occasionally to five or six years. The long-term behaviour of structures that incorporate timber and wood-based materials is therefore uncertain. The experimental evidence gained from this study suggests that particleboard under

sustained load in sheltered situations reaches a limit state beyond which no further stress relaxation or deformation will occur in ambient conditions when stressed to less than 0.3 of short-term ultimate strength. The experimental evidence suggests that clear wood, as distinct from structural grade sawn timber, also reaches a similar limit state under a sustained tensile stress less than 0.6 of short-term strength.

In establishing these limits, the duration-of-load factors currently used to account for stress relaxation during service are brought into question. Throughout the world, design stresses specified in timber structures codes are commonly based on the 50 year old Madison curve, which is now held to be inaccurate. This study points to the possibility that design stresses in sawn timber structures that support sustained loads in exposed situations are some 50% too high. The associated uncertainty in predicting long-term behaviour is exacerbated by the way in which the capacity factor is derived for use with the probabilistic design methods specified by AS 1720.1-1997.

With respect to the influence of probabilistic based design methods, the capacity factor quantifies probability of failure that is related to either small clear wood specimens or small sections of highly variable sawn timber. It fails to account for the impact of design and construction quality. Consideration of structure reliability associated with the low variability of the mechanical properties of particleboard and the impact of design and construction methods necessitated a re-examination of the capacity factor. This revealed the presence of an anomaly in its derivation whereby a calibration factor was introduced which implies that design strengths are in excess of test strengths. The anomaly, which carries unsafe connotations, is removed and the function of the capacity factor is clarified by subdividing it into components that account separately for design and construction variability and for structure importance and by removing it from evaluation of material properties. The subdivided capacity factor is applicable to all structures that incorporate structural grades of timber and reconstituted wood products.

On settling these matters, attention turned to demonstrating the feasibility of using particleboard as a primary element in heavily loaded structures. The mechanical properties of particleboard and the strength of nail fastened connections were established and a storey height Vierendeel type wall beam with particleboard shear diaphragms was designed, using the revised capacity factor and the experimentally determined 50 year duration–of-load factor, to support a load of nine tonnes over a span of nine meters. Subject to a prototype test load of twenty-seven tonnes the structure behaved elastically, and withstood ultimately load more than forty tonnes.

The findings of this study should enable particleboard to be included in the AS 1720.1 and complement plywood as a reliable sheet material suitable for general structural use in sheltered environments.

THE STRUCTURAL USE OF PARTICLEBOARD

CHAPTER ONE

INTRODUCTION

1.1 STUDY RATIONALE

This study is concerned with the development of the fundamental design basis and design data that will facilitate the structural use of particleboard. It is motivated by:

- Particleboard's low cost,
- the fact that it can be manufactured from an abundant renewable resource,

- the fact that it requires no new trade skills or high-powered tools for its application,
- the strong belief that it will lead to reductions in the cost of some aspects of building construction.

Particleboard is presently restricted to applications where it is loaded normal to the sheet surface in flooring, shelving, and occasionally, formwork. In the absence of any specific design methods and any published authoritive design data, application as shear diaphragms in structures supporting heavy loads is not normally undertaken nor is it widely accepted that it is feasible. The writer's experience over many years with the design and construction of timber structures that utilized reconstituted wood panels, provided further motivation for this study.

1.2 BRIEF HISTORY OF PARTICLEBOARD USE

Empirical structural design methods for large timber structures based on testing large sectioned timber beams and columns existed c1810. Further development was eclipsed by the introduction of wrought iron, c1820, steel c1860, and concrete c1880 and the contemporaneous development of elastic theory and truss analysis. Development slowed for a century or so until a looming war brought shortages of steel and renewed interest in timber structures in the 1930's. Australia was an early leader in the study of timber structures during this era with work by the Council for Scientific and Industrial Research, later the Commonwealth Scientific and industrial Research Organisation (CSIRO), and the Commonwealth Experimental Building Station (CEBS). Concurrent work by the Forest Products Research Laboratory (FPRL) in the United Kingdom and the Forest Service of the United States Department of Agriculture (USDA) saw a period when a deepening appreciation of timber mechanics developed. Research is now spread world wide.

Until relatively recent times, despite the premier position held by CSIRO, timber building construction methods employed in Australia generally lacked the structural sophistication commonly found in other industrialized countries. This evolved as a result of the Australian hardwood saw-millers' preoccupation with the use of green timber for stick construction in housing, which market consumes around half the timber milled. An unwillingness to dry hardwood scantling to a seasoned condition to avoid the difficulties associated with its large drying shrinkage, severely restricted the complementary use of wood-based materials in sheet form, (Taylor, 1968). In recent years, an improvement in the competitiveness of timber construction came about through systems that employed seasoned timber, wood-based panels and fabrication methods that reduced on-site costs. Largely forced on the Australian timber industry by the recent shift to concrete slab-on-ground construction, aided by a rising production of seasoned softwood scantling, this impetus brought in train a waning use of unseasoned hardwood and a widening use of seasoned timber and wood-based panels.

Introduced in Australia c1970, the use of particleboard for domestic flooring spread rapidly, displacing more costly timber strip flooring and plywood. Subsequently, in the absence of specified data in AS 1720.1-1997, *Timber structures, Part 1: Design methods*, the *Australian Building Systems Assessment Commission*, (ABSAC) in a retrospective action, issued an Opinion that allowed the specific use of flooring grade particleboard as domestic flooring. This Opinion, subject to triennial reviews, met the requirements of the regulatory authorities following which the *Australian Wood Panels Association* (AWPA) published safe load tables for particleboard flooring in 1985. However, the particularly good planar characteristics of particleboard remain largely neglected, the literature reflecting an overwhelming concern with behaviour under loading normal to the surface. The random chip orientation in commonly produced particleboard makes it virtually isotropic in the plane of the board. Compared with the orientation constraints associated with plywood and oriented strandboard, isotropy greatly simplifies design and construction when used as a shear diaphragm, which use exemplifies the widened application expected to result from this study.

Because the mechanical properties of particleboard and related duration-of-load factors are yet to be incorporated in Australian building regulations, such widened use is limited by the present dependence on ABSAC for the further approvals aligned with each specific application. Therefore, to support an extended structural use it is necessary to add to the Australian standard specification for particleboard, AS/NZS 1859.1-1997, *Reconstituted wood-based panels, Part 1, Particleboard,* values for characteristic planar stresses; ultimate tensile, compression and shear stresses. It is also necessary to add to AS 1720.1-1997 information covering climatic effects, duration-of-load effects and structural reliability together with suitable nail strength values.

1.3 STUDY OBJECTIVES

The aim of this study is to establish the fundamental design basis and critical data for the structural use of particleboard and demonstrate that the material behaves satisfactorily when used as a primary element in a full-scale structure. To achieve this aim, the following detailed objectives were established:

1. Evaluate the viability of particleboard as a structural material.
2. Undertake specific studies on the mechanical behaviour of particleboard where current information is inadequate, particularly its response to climatic conditions and long-duration loading.
3. Investigate the reliability of structures made with particleboard elements, as affected by variability in material properties and design and construction procedures.
4. Use the results of this research to develop a suite of design stresses, environmental modification factors and nail strength values that will facilitate the design of structures with particleboard elements.

5. Develop and evaluate a typical structure application in which particleboard acts as a primary element, namely, its use as a shear diaphragm in a long span heavily loaded wall beam.

1.4 STUDY OUTLINE

A comprehensive study of the literature demonstrated that, as normally manufactured for flooring, random three-layer particleboard is a viable structural material. It is sufficiently durable, dimensionally stable and robust, for general structural use in the normal protected building environment under appropriate levels of stress.

However, difficulty was experienced in drawing from the literature, factors that will reliably account for the reduced strength and increased deformation that occurs in service. The difficulty is exemplified by the differing values presently ascribed to these factors by other design codes. For example, *Eurocode 5, Design of timber structures, Development Draft ENV 1995-1-1: 1994*, and current drafts of BSS 5268: Part 2-1996, *Structural use of timber, Code of practice for permissible stress design, materials and workmanship*, specify factors that do not fit the literature and appear to encourage significant over-estimates of strength and under-estimates of deformation in service.

To overcome the difficulty, recourse was had not only to the strong qualitative It was observed that, rheologically, solid wood, plywood, hardboard and particleboard were similar, and that some timber and hardboard structures are performing satisfactorily after 35 years of service. As a result, a structure similar to one built in 1964 with hardboard shear diaphragms, (Refer Figure 8.1), was redesigned with particleboard diaphragms. Validation took the form of a successful load test on a truncated model of a storey height Vierendeel type wall beam with particleboard diaphragms that was designed to support a limit state design load of 10*kN/m* over a 9*m* span.

Two identical sections of the wall beam, truncated to suit the available laboratory floor space, were fabricated, in the field, by a skilled tradesman using normal hand-held tools. When subjected to the prototype test specified by AS 1720.1-1997, both withstood 4.8 times the limit state design load. To verify the truncated models' test procedure, further work followed on a true third scale model, 3*m* span, built with hardboard diaphragms that was similarly designed and tested. It too withstood better than 4.8 times the design load, with behaviour proportionally identical to that of the truncated models. The truncated models' performance was thereby validated as representing accurately the behaviour of the full size wall. A second validation was accomplished by constructing a finite element model of the wall. It too represented reasonably well the deformations measured on the truncated and third scale models.

However, it was noted that the prototype test loads prescribed by Eurocode 5 and BSS 5268-1996 were significantly lower at 3.5 and 2.9 times the same limit state design load respectively which raised a question over the appropriate value of the duration-of-load factor. Verification of this factor was obviously necessary. To obtain an appropriate factor for particleboard, extensive long-term relaxation tests were conducted in both indoor and protected outdoor environments and in a controlled environment chamber. The results verified the design assumption and enabled duration-of-load factors related to climate to be established for general structural design use.

Because nail fasteners are most commonly employed to field fabricate timber structures, design loads for connecting particleboard to timber were also determined. This work revealed that connections made with common steel wire nails have the same capacity as nailed connections made with timber of similar density to particleboard.

The foregoing is readily illustrated through reference to the various terms employed in the equations specified for estimating the design resistance and deformation of structures. The general expressions for strength limit states design resistance of timber structures as typically specified in structural design codes, and in AS 1720.1-1997 in particular, are

for members $$R^* = \phi R = \phi \Pi k f_0' X \geq S^* \quad (1.1a)$$

and for joints $$R^* = \phi N_j = \phi \Pi k n Q_k \geq S^* \quad (1.1b)$$

where R^* is the estimated resistance

S^* estimated load effects

ϕ capacity factor

R Characteristic member design strength

Πk product of factors that account for variability of strength in environment and structure / component configuration

f_0' characteristic 5%ile material stress appropriate to the failure mode

X geometric property appropriate to the loading mode

Q_k characteristic fastener strength

n number of fasteners

N_j characteristic joint design strength

The product of strength factors, Πk, includes factors for climatic effects and stress relaxation. Climatic effects are discussed in Chapter Three and the effects of stress relaxation, together with the effects of design and construction procedures and the impact of load effects on the value of ϕ are discussed in Chapter Four. Characteristic values for the strength of particleboard, f_0', and fastener strength in particleboard, Q_k, are established in Chapters Six and Seven.

The general expression for determining the deformation of a member is estimated in accordance with the complex function

$$\Delta_0 = f(j, Q^*, E_0, X) \qquad 1.2$$

where Δ_0 is the estimated deformation.

j creep factor that accounts for the period of application of the load effect and its environmental variability.

Q^* specific design load or design action effect.

E_0 characteristic mean short term elastic property appropriate to the deformation mode.

AS 1720.1-1997 makes no explicit reference to mechanosorption which is the major cause of creep. The investigation into mechanosorptive effects, discussed in Chapter Four, revealed a better view of creep that should bring more confidence to estimates of long-term of deformation in timber structures.

1.5 STUDY OUTCOME

As stated previously, the study demonstrates that, when situated in sheltered environments, reliable and durable structures incorporating primary particleboard elements are feasible in the context of normal limit states design methods, commonly used trade skills and construction methods.

The study established the following:

1) That particleboard, appropriately manufactured under adequate quality control, is suitable for general structural use. This is discussed in Chapter Two.
2) The mechanical response of particleboard to moisture change. This necessitates the introduction of climate related Service Environments. Their characteristics and regional delineation are given in Chapter Three.
3) Duration-of-load factors for estimating the long-term strength and deformation of structures incorporating particleboard. Values are tabulated in Chapter Four.
4) Capacity factors and design and construction factors suitable for timber structures generally together with a modified material characterization formula. Factors and formula are given in Chapter Five.
5) Characteristic mechanical properties of particleboard suitable for structural design purposes. Values are tabulated in Chapter Six.
6) Characteristic nail strengths for the design of connections between particleboard and timber. Values are tabulated in Chapter Seven.
7) The ability of particleboard to act as a principal element in heavily loaded structures. This is demonstrated in Chapter Eight.

The findings should enable particleboard to be included in AS 1720.1 and thus complement plywood as a sheet material suitable for general structural use.

CHAPTER TWO

A REVIEW OF THE LITERATURE

2.1 INTRODUCTION

2.1.1 Material Attributes.

A structural material must hold four fundamental attributes.

- That it is durable, retaining physical integrity and dimensional stability during the service life of a structure.
- That the manufacturing process produces material of consistent structural quality.
- That data is established which enables reliable structures with a service life of 50 years to be built.

- That it is workable and robust, requiring neither special installation techniques nor abnormal supervision to form reliable structures, and presents no hazard to health.

The following detailed review of the literature was made to establish whether, and under what conditions, the foregoing could be satisfied by structural particleboard.

It revealed that the first attribute is met readily. Physical integrity and dimensional stability are functions of several factors; the chemical stability of the adhesive, the quality of the adhesive bond between the cellulose fibres and the magnitude of the stresses induced, or relieved, by variations in moisture content. In all other respects, particleboard is as durable and stable as its constituent wood. However, unlike wood itself, which requires treatment against biological attack only to ensure indefinite longevity when exposed to weather or other moist conditions, the use of particleboard in such conditions is not feasible. Irrespective of the type of adhesive used in its composition or the addition of a preservative, particleboard loses all useful strength in situations where its moisture content exceeds 18% for prolonged periods. Only a totally reliable and effectively maintained impervious protective coating would obviate the mechanically debilitating effects of excessive moisture absorption.

The second attribute is also readily met. Particleboard quality is very sensitive to the characteristics of the component materials and processing methods. The manufacturing process exerts such tight control that the product displays a degree of variability appreciably lower than that exhibited by most structural materials. Coefficients of variation in mechanical properties between 5% and 10% are consistently achieved. A strong relationship between dimensional stability and strength exists so that dimensional stability carries with it consistent structural characteristics. Particleboard can be treated at low cost against biological degradation with the commonly used wood preservatives and without mechanical properties being impaired. Compared with the orthotropic nature of solid wood, the prime effect of reconstituting wood chips into an isotropic panel is to nominally double its deformation and halve its strength.

With respect to the third attribute, the literature revealed no qualitative difference between the behaviour of wood and reconstituted wood-based panels in response to environmental factors nor in their ability to perform satisfactorily for 50 years in sheltered situations. Creep and stress relaxation under sustained load increase significantly when they undergo changes in moisture content induced by ambient humidity variations. These phenomena are tied to sub-microscopic behaviour of the wood material caused by the movement of moisture, but experimental methods to examine the underlying mechanisms are not yet available. Several mathematical models exist, but it is clear that behaviour under load is yet to be fully explained. The duration-of-load factors are developed as part of this study to account for stress relaxation in particleboard. The experimental methods employed in their determination appear to remove some of the conjecture surrounding long-term behaviour. Obviously enough, the matters that influence the structural use of wood have come to be appreciated even though they are not fundamentally understood. Factors that account for the uncertainty inherent in the design of timber structures have been developed empirically through use over millennia; a position not entirely unique amongst the structural materials in use today. In this study, the capacity factors that reflect the reliability of limit states design methods are examined with the result that the lower variability of particleboard's mechanical properties is shown to reduce design and construction uncertainty.

Possession of the fourth attribute is amply demonstrated through its field use as flooring and lining during the past three decades. Sufficient robustness to withstand the handling normally met on a building site is implicit in its widespread use. Except for the need to protect particleboard from excessive moisture during construction, the skill levels and low energy tools and equipment employed to fabricate structures with particleboard elements are no different to those required for normal sawn timber construction. With respect to safety, properly handled, particleboard presents no hazard to health, because the level of formaldehyde emission is stringently limited and controlled by AS/NZS 1859.1-1997. Further amplification of these aspects of particleboard as an acceptable structural element is unnecessary so far as this study is concerned.

2.1.2 Description of the Material.

Particleboard is defined in AS/NZS 1859.1-1997 as *"a panel material manufactured under heat and pressure from particles of wood (wood flakes, chips, shavings, sawdust and similar) and/or lignocellulosic materials in particle form (flax shives, hemp shives, bagasse fragments, rice hulls, wheat straw and similar) with the addition of an adhesive."* Similarly, the British Standard BSS 5669-1989: Part 2: *Specification for wood chipboard*, and the British European Standard BS EN 309, define particleboard as *"a panel material manufactured under pressure and heat from particles of wood (wood flakes, chips, shavings, sawdust, wafers, strands and similar) and/or other lignocellulosic material in* *particle form (flax shives, hemp shives, bagasse fragments and similar) with the addition of an adhesive."*

In Australia, the common form of particleboard consists of wood, as chips or fibres, laid randomly and bound with various adhesives. It ranges in density from 600 to around $800 kg/m^3$, most commonly 650 to $720 kg/m^3$ and in thickness from $6 mm$ to $50 mm$, mainly 12 to $19 mm$. The manufacturing process involves the flaking of wood into pieces in the order of $20 mm$ long x $10 mm$ x $0.4 mm$ which are then dried to a low moisture content, around 4 or 5%, and mixed with glue before being formed into a mat. The mat is then simultaneously compressed under some $4 kPa$ and heated to about $120°C$ while the glue gains an initial set. Upon release from the press, the resulting $2.4 m$ wide and $10 m$ long sheet is trimmed to size and sanded. Generally the intermittent presses employed are multi-decked, producing between four and twelve boards in a cycle. Some board at the $6 mm$ to $8 mm$ end of the thickness range is made in continuous-action roll presses where the particles tend to become oriented to a minor extent across the board, as opposed to the more commonly used intermittent pressing where the tendency is to minor orientation in the forming direction.

Forestry agencies throughout the world attempt to reduce the quantity of wood fibre left in the forest after harvesting, reckoned roughly to equal the amount removed, while the milling of the wood taken creates yet another nominal 50% waste, resulting in a nominal 25% recovery. Particleboard, by its very nature, appears to be a means of utilising some of this waste material and the effect of incorporating various forms of milling and processing residue on quality and production is the subject of continuous research. Around 50% of the furnish in particleboard plants originates as saw-mill and planer-mill residue, and the local use of waste and residue wood could be expected to increase as a means of reducing cost. It may also be expected that various types of agriculturally sourced lignocellulosic fibre, such as bagasse and wheat straw, will be utilised in the near future.

2.1.3 Research History.

Particleboard has been made since the mid-1950's and its literature reaches back to 1960 when the industry was in its infancy, while the literature on adhesives follows the growth of the petroleum industry. Investigation into the mechanical nature of particleboard was undertaken by many researchers spread over numerous institutions in Europe and the United States and more recently in Australia. Because investigations were made into the effects of many timber species and types of adhesives held under various regimes of time, temperature, humidity, and stress, the literature lacked coherence and made interpretation difficult. It was evident that parallels between the characteristics of timber and reconstituted wood-based panel products had to be drawn so that the physical and environmental affects particular to particleboard could be extracted. A subdivision of the literature into independent sections is attempted, but could not be regarded as entirely successful because many aspects of its behaviour are interdependent.

2.2 DURABILITY

2.2.1 Introduction

The literature was examined to establish whether particleboard would retain its physical integrity and dimensional stability over a fifty or sixty year period. Having been made for a bare 30 years in Australia, an assessment of durability may only be based on its similarity to wood and other wood-based materials, such as glue laminated timber, plywood and hardboard, which all have longer histories of 50 years or more. It was evident from the literature that the physical deterioration effected by moisture on the wood-fibre bond was at the core of particleboard's durability. Consequently, reductions in the quantum and rate of moisture absorption by the board also reduced the rate by which mechanical properties were impaired over the long term. Suitable test methods to assess long-term physical integrity and dimensional stability have been developed.

2.2.2 Biological Degradation

Given appropriate conditions, all wood and wood composites suffer biological attack. The Wood Handbook (1974) summarised the susceptibility of wood to biological attack as follows:

"Attack essentially ceases when temperatures lie outside the range 2°C to 38°C. Serious decay only occurs when the moisture content of the wood is at fibre saturation point which is in the order of 30% depending on the species. The water vapour in humid air alone will not wet wood sufficiently to support significant decay but it will permit development of some mould. Fully air dried wood usually will have a moisture content less than 20% and should provide a reasonable margin of safety against fungal damage. Thus wood will not decay if it is kept air dry." The biological degradation of particleboard can also be caused by fungi and termites, with the rate of degradation dependent on wood species, type and amount of adhesive, size and position of particles, surface hardness and the humidity of the surrounding air. CSIRO (1962) stated that *"both urea and phenol-formaldehyde glues possess good fungal resistance and phenolic glues additionally resist bacteria"*.

Toole and Barnes (1974) reviewed the literature on the biological deterioration of particleboard bound with formaldehyde adhesives, and followed this up by soaking untreated board in water for 24 hours after which it was noted that damage by both fungi and termites had significantly increased. It was concluded that soaking at room temperature caused materials to be released within the board that hastened biological degradation. However, they verified that termite and fungal attack could be controlled by the addition of toxic compounds to the adhesive. It was also noted that both urea and phenolic bonded boards fared equally poorly in their loss of mechanical strength after treatment. Therefore, Toole and Barnes concluded that particleboard made with either of these adhesives would require suitable biological protection, in common with solid wood, when relative humidity approached 100%, i.e., when its moisture content exceeded 20%. By virtue of their low viscosity, low molecular weight formaldehyde resins penetrate the cell wall readily, reduce capillarity and slow the rate of moisture absorption. Due to the bulking effect of the adhesive, the equilibrium moisture content of particleboard in the normal ambient range was always less than that of solid wood. It followed that particleboard was less susceptible to biological degradation than solid wood under the same conditions.

Rowell and Ellis (1981) continued Rowell's (1980) investigation into biodegradation by examining the reduction in attack caused by the bulking that isocyanate adhesives provided by virtue of their total chemical bonding to both the cellulose and lignin. By measuring the amount of the isocyanate compound bound to the wood fibre, and studying its distribution within the lignocellulose, they showed *"that 60% of the lignin hydroxyls were substituted and 12% of the holocellulose hydroxyls were substituted at the point where resistance to biological attack occurred"*.

2.2.3 Surface Treatments

In attempts to improve the durability of particleboard, numerous researchers examined surface treatments as a means of reducing moisture absorption. Gatchell et al (1966), applied paint, water repellents and overlays to particleboard which, when judged by the swelling in thickness after exposure to weather for several years, showed paint to be the surface treatment most effective in reducing water absorption. The oxidation of the paint surface heralded the onset of an increased rate of deterioration by allowing moisture to permeate more readily, and drew attention to the need for a high level of maintenance to prevent rapid deterioration when exposed to weather continuously. They also noted that painted boards and unsanded, partially glue-covered boards, both of which exposed less wood surface to weather, showed a much reduced tendency to swell compared with the swelling induced by accelerated tests. Knight (1968) investigated painted plywood bonded with an unfortified urea-formaldehyde resin which readily hydrolised in the presence of moisture. Following exposure for 10 years to the various marine environments found in a boat, among them exposure on the deck and in the bilge, it was shown that a properly maintained paint coat would completely prevent deterioration. Lehmann (1972) examined moisture effects in boards bound with various adhesives and concluded that "*the two best possibilities for panel stability still appear to be coatings and bulking agents which prevent the passage, and/or subsequent absorption of liquid or vapour water provided that optimum particle lengths and bonding conditions are present*". He found that linear expansion was a direct function of the amount of moisture absorbed.

2.2.4 Bulking Agents

To slow moisture absorption, various researchers examined the addition of cell wall bulking agents in detail. Gatchell et al (1966) studied the affects on durability of the variables in the manufacturing process by exposing control samples to weather for one year at a number of sites throughout the United States, and then compared them with their matched counterparts after an accelerated ageing test. The addition of around 1% of paraffin wax was found to significantly increase strength and dimensional stability, but in excess of 1%, improvement was considered marginal, possibly because of the greater loss of adhesion that would accompany the increased number of wax-coated wood particles. Hann et al (1963) showed that boards with no wax lost practically all strength within a year. When waxed particleboards bound with urea-formaldehyde were exposed to weather nominally half the deterioration in mechanical properties took place in a year and most of it within two. Boards made with a phenolic resin suffered a reduction in elasticity similar to those with a urea-formaldehyde resin, but a somewhat smaller loss of strength. Hall and Gert Je Jansen (1974) found that the addition of linseed oil reduced swelling.

Hunt (1976) confirmed the foregoing with tests on board bound with a mixture of urea-formaldehyde, 40% and phenol formaldehyde, 60%, that had been exposed for up to six years in London to both typical indoor and sheltered outdoor conditions. One of the board types examined contained no wax and its markedly different behaviour caused Hunt to examine the response to humidity changes induced by the wax. After being conditioned at $23°C$ to $35\%rh$, several boards were exposed for 24 hours to $65\%rh$ at the same temperature, following which the moisture absorbed was determined and Fick's diffusion rate computed. Board with 0.5% wax was found to be about one-fifth as permeable as unwaxed board. Hunt considered that a minimum wax content of 1% was necessary to substantially reduce the rate of moisture absorption and mitigate adverse mechanical response to humidity fluctuations. Rowell and Ellis (1981) believed that isocyanate adhesives have a similar effect due to their chemical bonding with both lignin and cellulose.

2.2.5 Accelerated Ageing

There is evidence that accelerated tests produce bond failures that may not be realised in actual service environments. For example, Gillespie and River (1976), after exposing plywood bonded with melamine-formaldehyde on a test fence for seven years, were unable to detect the tendency to hydrolyse that was found with accelerated tests. In part, the explanation was related to the higher stresses developed by the relatively sudden volumetric changes produced under test compared with the normal environment, where stresses in both the wood particles and the adhesive would be blunted by relaxation over a much longer period.

Hann et al (1963) endeavoured to develop a relationship between short term and long term properties and accelerated ageing for both urea and phenol boards. The tests conducted subjected the samples to soaking in hot water and steam for various periods as under:

(a) Water soak at $49°C$ for $1hr$.

(b) Steam spray at $93°C$ for $3hrs$.

(c) Store in air at $12°C$ for $20hrs$.

(d) Dry in air at $99°C$ for $3hrs$.

(e) Steam spray at $93°C$ for $3hrs$.

(f) Dry in air at 99°*C* for 18*hrs*.

Following six cycles of the above regime, the specimens were returned to equilibrium at 24°*C* and 65%*rh* before mechanical properties were re-measured. It should be noted that this test would take 12 days to complete and later became the accelerated ageing exposure test adopted by the American Society for Testing Materials (ASTM) in test method ASTM D-1037. The ageing was found to correlate reasonably well to the properties retained after three years of exposure for phenolic bonded boards, but the tests predicted a greater degradation for urea bonded board than was observed when actually exposure to weather. Under the foregoing regime the particleboard would be heated for a further period, and it was considered that the additional curing of the adhesive provided by the test heat would obscure the lesser, but actual, bond strength which existed in the board at the time of manufacture. A possible further factor was an aggravated chemical decomposition induced by the heat energy added by the test.

Laidlaw and Beech (1973) also developed an ageing test method that used three repetitions of the following regime, and took 21 days to complete:

(a) Soak in water at 20°*C* for 72*hrs*.

(b) Freeze in air at -12°*C* for 24*hrs*.

(c) Dry in air at 70°*C* for 72*hrs*.

After the specimens were reconditioned to 20°*C* and 65%*rh*, properties were reassessed and found to correlate closely to those retained after exposure to weather for two years. It was concluded that internal bond strength was the most reliable indicator of the capacity of a board to remain stable and retain its physical integrity. McNatt, Lehmann, and others subsequently confirmed this view. The V313 test, prescribed by AS/NZS 4266.11-1995, *Reconstituted wood-based panels-Methods of test*, is essentially due to Laidlaw and Beech, and also occupied 21 days. It consisted of 3 cycles of the regime, as under:

(a) Soak in water at $20°C$ for $72hrs$.

(b) Freeze in air between $-12°C$ and $-20°C$ for $24hrs$.

(c) Dry in air at $70°C$ for $72hrs$.

The writer performed wet cyclic internal bond tests on bagasse particleboard bound with isocyanate for the Monash Timber Engineering Centre (1997), and found that it readily satisfied the V313 test.

Steiner and Chow (1975) studied the performance of resorcinol-phenol-formaldehyde resins in plywood at extremely high and low temperatures. They found that the adhesive's bond strength had not been reduced under the following vacuum-pressure-soak regime which occupied 8 hours.

(a) Water soak $2hrs$. at $10°C$ at $-85kPa$

(b) Water soak $2hrs$. at 10°C at + $550kPa$

(c) Prior to testing, condition in air for four hours at $12°C$ (Test 1) and $65°C$ (Test 2).

Their investigation was extended to embrace bonding at temperatures below −60°C with urea, melamine-urea, and phenol formaldehydes, polyvinyl acetate, and casein adhesives. Irrespective of the type of adhesive, wood failure was found to be high with all adhesives and occurred in the region immediately below the glueline. The reason was not discussed.

2.2.6 Formaldehyde Adhesives

Several adhesives are in worldwide commercial use to make particle and fibreboards, most commonly the various formaldehydes which have been under continuous development since the mid-1930's. Of the adhesives employed for board manufacture, non-waterproof urea-formaldehyde enjoys the widest use. Among others, all regarded as waterproof, were the phenol and resorcinol-formaldehydes and a limited amount of tannin-formaldehyde, the latter widely used in South Africa. However, not only are the waterproof adhesives more expensive than urea-formaldehyde in first cost, but their longer curing periods increase production costs by lengthening the press cycle and by making process control more difficult due to their greater chemical reactivity. To improve process control and thereby reduce cost, melamine-urea-formaldehyde mixtures were developed. These mixtures, while not waterproof, are highly water-resistant and cost no more than the fully waterproof adhesives, and retain processing characteristics similar to the more manageable straight urea-formaldehyde.

Blomquist and Olsen (1964) investigated plywood bound with urea-formaldehyde fortified with differing proportions of melamine-formaldehyde, and after exposure to weather for 12 years found that once the melamine proportion exceeded about 20%, approximately half the glue line shear strength was retained. With proportions between 20% and 80% the loss was held to around 50%, and as proportions exceeded 80% strength started to improve again. Selbo (1964), in the search for an accelerated test that would readily differentiate durable from non-durable glues, obtained somewhat better results when investigating glue laminated timber held under exterior service without any protective coating. It was observed that unfortified urea-formaldehyde resin was non-durable, but a 50-50 mixture of melamine and urea-formaldehydes was the equal of recorsinol-formaldehyde. The Wood Handbook (1974) in Table 9.2 referred to melamine resin as being very resistant to moisture and damp conditions depending on the type and amount of catalyst used.

Laidlaw and Beech (1973) examined urea-formaldehyde, melamine-urea-formaldehyde, phenol-formaldehyde and sulphite liquor for their respective quality of adhesion with the forerunner of the V313 test method referred to earlier. Phenol boards recorded losses in internal bond and modulus of rupture of less than 25%, but a severe loss of 40% in modulus of elasticity caused them to *"suggest caution in predicting any satisfactory performance in exterior situations for periods in excess of five years"*. These losses were practically identical to the findings by other researchers and it was concluded that the optimum amount of phenol-formaldehyde that could be used was probably 12% in view of the increasing manufacturing difficulties and higher costs that larger proportions created in return for a relatively small gain in strength. Earlier, Gatchell et al (1966) found that a transition from surface failure between particles, to an in-particle failure, occurred with a resin content of about 6% beyond which little extra strength was gained.

While urea-formaldehyde dissociates completely and fairly rapidly when exposed simultaneously to moisture and a low level of heat, it is totally resistant to hydrolysis in cold water. The actual level of tolerance to combined heat and moisture has not been established, but it is generally agreed that unfortified urea-formaldehyde will not hydrolyse over the "long-term" within the interior of buildings. The low resistance of urea-formaldehyde to elevated temperatures, such as may exist in some roof spaces during hot weather, was referred to in a Technical Note by CSIRO (1962). As opposed to this, glulaminated timber building structures using urea-formaldehyde fortified with melamine formaldehyde have performed satisfactorily in Europe for fifty years or more.

The Wood Handbook, (1974) considered phenol or resorcinol-phenol-formaldehydes to be *"more resistant than wood to high temperatures"*. Having regard to use in all weather, humidity, temperature and applied stress conditions, CSIRO expressed the view that because the more reactive recorsinol-formaldehydes provided an erratic quality of bonding in finger joints in solid wood, the comparatively more manageable phenolic resins were *"the most acceptable binders"* whether for particleboard or any other reconstituted wood product. They also argued that preference should lie with dark coloured phenolic resins for structural use because, being the same light colour as urea formaldehyde, it would not be possible to visually detect the presence of melamine in melamine-urea-formaldehyde. However, the field experience embodied in European codes allows the use of any adhesive provided satisfactory internal bond strength is retained after accelerated ageing by the V313 test.

CSIRO (1962) believed that tannin-formaldehyde was a very durable particleboard resin suitable for external use because it was chemically very stable. A limited amount of particleboard has been made in Australia, for 'wet area' application in houses, using tannin formaldehyde mixed with a small amount (2%) of recorsinol formaldehyde to shorten press times. No subsequent release of formaldehyde occurred under ambient conditions.

2.2.7 Isocyanate Adhesives

Developed by Bayer in the mid-1930's, polymeric dimethyldiisocyanate compounds have been made specifically for wood based panels since 1960. Its use for particleboard and fibreboard manufacture only became possible recently through the development of release agents which prevented adhesion to the press platens. Isocyanate reacts with free hydroxyl groups in both the lignin and the holocellulose comprising the wood cell walls and form true insoluble molecular bonds directly with the lignocellulose. Bonding with the cellulose is a completed chemical reaction so that the resulting board is no less waterproof than solid wood. A very low viscosity at normal processing temperatures enabled isocyanate to readily permeate the cell walls of the wood thus enhancing adhesion quality. An important associated virtue of their low viscosity was the bulking effect created in the holocellulose in particular, which was found by Rowell (1980) to provide very good protection against biodegradation. Yazaki (1966) considered that good housekeeping in the glue preparation area would avoid any toxicity problems in the manufacturing plant, and that the toxicity of gases released on burning during a fire would be far overshadowed by the amount of carbon dioxide generated.

Brynyldsen et al (1976), when discussing further research needs for structural particleboard, referred to isocyanate adhesives as *"particularly promising (in) giving great stability at moisture changes and comparatively slight creep"* when compared with urea or phenol bonded board. Ball and Redman (1978) also demonstrated that isocyanate was a most effective binder for wood, and that it provided a superior resistance to thickness swell. Yoshida et al (1994), in examining the bond quality and long term structural performance of isocyanate adhesives, concluded that the bond durability was the equal of resorcinol-formaldehyde. Isocyanates were included recently in the German standard DIN 68 763 (9/90) as suitable for all types of particleboard. As referred to earlier, the thickness swell and glue bond quality of isocyanate bonded particleboard, as determined by the V313 test method set down in AS/NZS 4266.11-1995, met the requirements of AS/NZS 1859-1997 no less satisfactorily than particleboard bonded with phenol or melamine fortified ureaformaldehyde.

2.2.8 Chemical Degradation

Wood has been shown to be highly resistant to weak acid and alkalis, The Wood Handbook (1974). Should the resistance of the adhesive be at least that of wood itself, it then followed that particleboard would be sufficiently resistant to chemical degradation for any structural use for which wood was appropriate, provided concentrated acids and alkalis were not met. CSIRO (1962) stated that phenol-formaldehyde resists weak acids and alkalis. The Wood Handbook (1974) also referred to the acid resistance of wood being improved by impregnation with phenolic resins.

No reference to electrolytic action and the corrosion of metal fasteners in wet particleboard has been found, but the action in wet timber has been investigated by Baker (1974) who concluded that, in the absence of moisture, mild steel fixings do not corrode in timber. Stainless steel, monel and plastic fixings are unaffected by moisture.

The chemical action of the various common wood preservatives on the gluelines of laminated timber was examined by Selbo (1967) and Jokerst (1972) independently. Treated glulam was subjected to weather for 20 years, and for all practical purposes no deterioration could be observed. Jokerst (1975), in continuing this investigation into the same glulam, exposed by then to weather for 23 years, found that, where an oil vehicle had been used to transport the preservative, the urea-formaldehyde gluelines had largely remained intact, far exceeding their expectations. But, when treated with waterborne preservatives, water had reached the glue lines and all glulam bound with urea-formaldehyde had failed completely.

2.2.9 Stress Degradation

The mechanism of the breakdown of the wood-glue bond under stress is not understood. It is thought to occur either at the wood-glue interface or within the wood immediately below the interface rather than in the glue itself. The sites at which true adhesive bonding failures can be identified are located in the cellulose of the cell wall, not in the lignin that binds cells together. Halligan (1970) noted that the larger the swelling of particleboard during the uptake of moisture, the larger the initial creep and the poorer the mechanical properties. Halligan and Schniewind (1972) subjected urea bonded board in bending to cycles of humidity from 30%*rh* to 97%*rh*, i.e., 5.5%*mc* to more than 20%*mc*, and found that creep was directly proportional to thickness swell. Thus a limit on thickness swell, as specified by AS/NZS 1859-1997, could be regarded as an important indicator of mechanical properties and a useful process control tool.

According to Christensen (1975) *"boundary layers need only be a few molecules thick to develop their own different mechanical properties and become a distinct structure component in the sandwich"*. Weak boundary layers are formed where the chip surface is covered in part with dirt, grease, or atmospheric condensates and the natural resins and waxes found in wood, which, together with the water repellents added to the board furnish, only compound the problem. Further, in chipping the wood to make particles, the wood surface is subjected to mechanical damage, and fibres so made on the chip surface may themselves not be strongly bound to the chip. In an endeavour to separate the specific effects of swelling and shrinkage of the wood fibre from overall strength losses when under load, Christensen (1975) bonded vermiculite to urea-formaldehyde and found that no loss of strength occurred under moisture cycling. This suggests that moisture changes in the wood alone set up the stresses responsible for the strains that had created the checking and lowered internal bond strength.

Christensen subsequently, (CSIRO 1977), examined the effects that large moisture changes had on urea bonded particleboard by cycling relative humidity between 25% and 90% at 38°*C*, which gave board moisture contents between 5% and 14%. After 70 cycles, extensive internal checking and surface looseness had developed accompanied by a significant reduction in internal bond strength, all attributable he believed, to the failure of molecular bonds in the glue as a result of the strains developed by moisture changes in the wood particles. Christensen went on to examine the adhesion formed between formaldehydes and wood, and concluded that "*mechanical bonds of moderate strength between the adhesive and the wood component could result from frictional interlock alone*". His contention being that the development of true molecular bonding to the wood surface would occur but occasionally. Christensen did not examine the total chemical bonding formed between isocyanate adhesives and the ligno-cellulose. Hunt (1976) similarly expressed the view that the loss of strength in urea bound particleboard may be due not only to hydrolysis of the resin, but also to the stresses between particles caused by the swelling and shrinkage induced by moisture changes. And further, that the loss would be enhanced by the consequential irreversible expansion when the compression set impressed on the wood particles during manufacture was released. The stresses referred to above are discussed in Section 2.3 on manufacturing variables.

In an early Technical Note (1962), CSIRO questioned the long-term durability of urea-formaldehyde under sustained stress; but this now contradicts the general thrust of the literature. The work by Selbo (1967) and Jokerst (1975) on glulam exposed to weather for 25 years, referred to above, may also be related to particleboard behaviour, in that their observations pointed to glueline failures as being either total, due to hydrolysis, or absent altogether. Block shear tests on the gluelines of the remaining sound glulam showed that failures which did not occur wholly within the wood were infrequent. This indicated that where the wood-glue bond itself had retained its integrity, the stresses set up by wet-to-dry differential swelling and shrinkage between the laminates were not large enough to damage the bond at all. Borgin (1972) too, examined glue bonding to solid wood, and concluded that there was *"no simple glue failure on a microscopic or submicroscopic level"*. Borgin considered that failure of molecular bonds in the adhesive itself was unlikely when properly formulated and cured, and more likely to be due to delamination of the wood cell walls. Christensen (1975) reached a similar conclusion. Dinwoodie (1978) holds that *"in the dry state there appears to be remarkably little difference in strength and stiffness among a wide range of particleboards"* and believed that, provided moisture contents remained less than 10%, particleboard made with any adhesive would retain its structural integrity indefinitely.

2.2.10 Discussion

Considerable research into the durability of particleboard stretching back 40 years or more, demonstrates that, excluding biological attack, durability is solely a function of the cellulose-to-adhesive bond. Appropriately formulated particleboard bound with any of the formaldehydes is sufficiently durable to maintain physical integrity for at least 50 years in the normal protected interior environment. The mechanics of the adhesion formed between formaldehyde resins and wood fibre has yet to be explained because the difficult experimental techniques needed to investigate the phenomena are not yet developed. Little literature on adhesive dispersion at the submicroscopic level has been found.

Particleboard made in Australia uses various formaldehydes and the evidence suggests that formaldehyde adhesion exists as a frictional bond only, resulting from mechanical interlocking at random sites where the adhesive penetrates irregularities in the surface of the wood fibre. The large majority of the failures are at the bonding sites in the wood fibre rather than in the glue mass; a conclusion supported by the direct relationship found to exist between all other mechanical properties and internal bond strength. However, the mechanism of failure in particleboard under stress is in part related to weaknesses in the boundary layer formed between the chip and the adhesive with failure observed to occur in the wood fibre rather than the adhesive. The quality of adhesion is no different between one type of formaldeyde or another and mechanical behaviour is independent of their respective strengths when board moisture content, on average, remains below 12%. Average moisture content would not exceed 12% under ambient conditions in a protected situation where free water was absent. But, in the presence of free water at ambient temperatures, unfortified urea-formaldehyde resins hydrolyse over time. Consequently, particleboard bound with urea-formaldehyde could be rendered potentially unsound by over-exposure to weather during construction and any other bad building practice that allowed the subsequent accumulation of free water deposited by moisture penetration or condensation. The use of waterproof adhesives does not mitigate the loss of strength associated with high moisture contents.

Phenol, resorcInol, and tannin-formaldehydes are totally water proof due to molecular cross linking which continues to improve with time, and all produce particleboard suitable for use in the normal protected building environment. A 1:1 mixture of melamine-formaldehyde and urea-formaldehyde removes absolutely any risk of hydrolysis at low levels of moisture and heat, and research indicates that in ambient conditions the durability of this adhesive mixture is equal in longevity to waterproof adhesives. One objection to melamine resins rests with their being the same light colour and therefore visually indistinguishable when mixed with urea-formaldehyde, whereas the waterproof adhesives are a dark, readily discernible colour. The objection is readily overcome by adding a colouring agent or marking the board surface as typically specified in BSS 5669.2-1989, *Particleboard.* As a general rule, it appears that the more waterproof a formaldehyde resin became, the more costly and difficult the manufacturing process.

On the other hand, totally waterproof isocyanate adhesives exhibit the amenable process characteristics of the urea-formaldehydes, and appear to be the equal of waterproof and water resistant formaldehydes, and therefore no less durable than wood fibre. Isocyanate adhesives form chemically stable, true molecular bonds with both the lignin and the cellulose in the wood fibre, so that friction and surface tension within the matrix is far less dominant in the development of strength compared with formaldehydes. Consequently isocyanates exhibit adequate adhesion for structural use. They have yet to be introduced commercially in Australia.

Records of relative humidity and temperature within buildings in Australia show that particleboard moisture content normally spans a 6% to 11.5% range, exceeding 11.5% rarely because a prolonged combination of 80%*rh* and 30°*C* is an infrequent occurrence. The delineation of the regional contour where this combination occurs is plotted on Fig. 4.1, and it is apparent that most building work falls under this contour. Beyond this temperature/humidity range, rapidly increasing loss of strength renders particleboard unsuitable for structural work irrespective of its constituent adhesive or the standard of construction.

Particleboard intended for structural use should ideally be assessed by tests in which temperatures were nominally ambient, so that the measured properties would represent the normal in-situ condition. Therefore, tests of glue bond quality and thickness stability should be carried out at room temperature with methods that require no heat except that drawn from ambient conditions. No such methods have yet been developed, and assessment of an adhesive may perhaps be judged best by actual field performance. The ability of accelerated ageing to truly represent service behaviour has long been questioned, and the evidence suggests that the V313 test as specified by AS/NZS 1859-1997 would create far greater loss of physical integrity than that developed during 50 years of service in a protected environment. This test requires not less than 45% of short-term-ultimate strength to be retained.

Only limited investigation into the effects of low temperatures on particleboard have been made and the behaviour of glulam in sub-zero conditions may provide the best information. Glulam bridges have been in service on logging roads in the high country of the United States of America and elsewhere for decades and it could be expected that failures attributable alone to adhesives subjected to sub-zero temperatures would be reported in the literature. None was found, and it is concluded that particleboard could be used in any sheltered environment in which wood can be used.

As with solid wood, a high moisture content increase particleboard's susceptibility to biological attack, irrespective of adhesive type, and preservative treatment may be necessary for biological protection against frequent condensation, water penetration, or prolonged high humidity. These conditions arise only with extended exposure to weather of unpainted board or want of protection from excessive condensation; conditions that reflect bad building practice quite unsuited to structural work. Neither strength nor elasticity is adversely affected in the long term by any of the wood preservatives in common use.

Dowel type fasteners normally used with solid wood are equally suitable for use with particleboard, regardless of the type of adhesive used in manufacture. Chemical degradation occurs only with corrodible fasteners exposed to weather, free water, or very high humidity; in conditions unsuited to the structural use of particleboard. Corrosion of steel fasteners does not occur in wood with a moisture content less than 15% and is less likely with particleboard in a sheltered environment due to its lower permeability and equilibrium moisture content. Therefore, common steel nails are capable of providing reliable connections to particleboard elements in a sheltered environment.

Structural connections of acceptable reliability are achievable using water-resistant adhesives applied in a workshop where temperature and humidity are closely controlled. However, before field gluing could be contemplated, satisfactory quality control measures, presently non-existent, would need to be incorporated in relevant codes of practice before the misgivings of engineers and building regulators with regard to its structural reliability could be allayed.

2.2.11 Conclusions

Provided its moisture content is kept below the level at which the adhesive hydrolyses, particleboard exhibits biological and chemical degradation similar to wood and is equally durable. Formaldehyde and isocyanate adhesives are effective binders in a protected environment. A reliable accelerated ageing test that exaggerates the loss of physical integrity that occurs when exposed to excessive moisture is specified in AS/NZS 1859.1-1997 in the form of the V313 test. A structurally acceptable certainty, therefore, attaches to the retention of physical integrity in excess of 50 years for particleboard that satisfies this test and is sheltered from sunlight and excessive moisture.

Properly formulated and maintained paint systems will protect particleboard bound with urea-formaldehyde from excessive moisture absorption as attained on direct exposure to weather, condensation, or water penetration in poorly constructed buildings. However, particleboard bound with urea-formaldehyde would require abnormally high maintenance of the paint system to retain physical integrity.

Structural particleboard bound with colourless melamine-formaldehyde and melamine fortified urea-formaldehyde adhesives needs appropriate identification to distinguish it from non-structural board bound with colourless straight urea-formaldehyde.

Tensile strength normal to the surface, called internal bond strength, is a reliable indicator of the level of mechanical properties possessed by particleboard. Minimum values of internal bond strength are specified by AS/NZS 1859.1-1997.

In a sheltered situation, reliable connections can be made to particleboard with the steel dowel fasteners in common use or with suitable adhesives applied and cured under workshop controlled conditions. The use of field-applied adhesives as the primary means of load transfer would be unacceptable except under skilled supervision.

2.3 MANUFACTURING

2.3.1 Introduction

The structural engineer's interest in manufacturing stems from the need to understand the effects of manufacturing variables on mechanical properties, so that control measures may be specified to maintain the quality needed for structural use. From this standpoint, the more readily can a product be made, the more acceptable must it become, because implicitly there is less room for unpredictable behaviour due to manufacturing error. The literature revealed a gradual improvement in the manufacture of particleboard as a result of an enormous amount of product research over the past four decades by numerous investigators. This research is ongoing. One underlying aim of the industry is to make board from as much of the available wood source, including wood waste, as possible. Consequently, the effects of incorporating in the particleboard furnish, bark, branches, dead wood, planer shavings, plywood and particleboard waste, and more recently, agricultural waste fibre, have all been studied and their impact on production costs and processing techniques is now well understood. Other manufacturing variables which affect processing, among them component moisture contents and press temperatures and pressures, have been thoroughly examined due to their major impact on processing time and costs.

The manufacturing variables reported in the literature cover the broadest possible range of effects due to

- Wood particles: Species, waste wood and bark, particle shape, size, and moisture content,
- Adhesive: Type, quantity, dispersion, curing temperature, curing period, forming pressure, chemical reactivity, water content,
- Production Process: Average density, density gradients, angle of grain in the wood particle, particle alignment in the board, mechanical damage to the wood particle, wax content, pesticides.

The effects of these variables on mechanical properties and quality control are discussed in the following sections.

2.3.2 Wood Particles

Lehmann and Geimer (1974) investigated the effects of combining three species of softwood and found that mechanical properties remained fairly constant with no obvious dependence on the mix proportions. The principal effect rested with the reactivity of a species to the adhesive and the affects on curing rate, and hence on press cycles and temperatures. The incorporation of waste wood and bark was also investigated. Linear expansion was similar for various proportions of bark, dead wood, and decayed wood, but the use of branches, in particular small branches, increased expansion six or seven fold. Thickness swelling of board with branch-wood was similar to that with other waste wood, and to the control sample of sound wood. Decayed wood produced board with a thickness swell some 20% less than the decay-free control board. Moduli of rupture and elasticity compared favourably with that of the control to the proportion where one half was sound wood, and the balance was wood about 50% decayed. Beyond this ratio, properties reduced in level. Evidently, adequate mechanical properties for practical use could be maintained provided the addition of decayed wood, bark and branch-wood was limited.

Coutts et al (1996) mixed several species of Australian hardwood obtained as thinnings, 15 to 17 years in age, to make particleboard and medium density fibreboard. The boards, all with densities close to $750 kg/m^3$ and bound with 8% of phenol-formaldehyde, were processed in similar conditions with regard to curing periods, press pressures and temperatures, as those which currently prevailed in Australian particleboard plants based on softwood. The mechanical properties of the particleboard and medium density fibreboard readily met AS/NZS 1859-1997 with internal bond values between 1050 and $1400 kPa$, and 200 and $320 kPa$ respectively.

No useful reference to mixtures of hardwoods and softwoods pertinent to structural use was found in the literature.

2.3.3 Particle Shape, Slope of Grain, Board Density, and Adhesive Content.

Lehmann (1973) examined the affects that the principal variables of adhesive content, particle geometry and board density, have on mechanical properties. It was found that all properties improved as density and adhesive content increased. An increase from 3% to 6% in glue content added some 30% to strength, while an increase in density of 15% increased strength and elasticity by a similar amount. With respect to dimensional stability, the effects of glue content and density were secondary compared with flake geometry. For example, linear expansion of board made with 10*mm* long flakes was double that of board with 50*mm* long flakes, while a 50% increase in flake thickness added about a third to thickness swelling. In general, a decreased flake thickness improved strength, while the length of a flake appeared to have virtually no influence on strength. The most significant factor found to improve stability was an increased glue content, followed by reductions in flake thickness and board density, both of which produced less swell. The improved stability was considered by Lehmann to be associated with the more ready relief of the compression strains impressed on smaller and more flexible particles during processing, and with the greater and more rapid response to moisture changes in less dense board. These general conclusions were similar to those drawn by McNatt (1973) and Hunt (1976).

Heebink (1975) examined the effect of slope of grain in the wood particles on properties using standard ASTM short-term tests. All boards showed the same characteristics as solid wood, that is, as slope of grain increased, so strength and stiffness decreased. Modulus of elasticity nominally halved from around 6000*MPa* for straight grained chips to 3500*MPa* for 1:5 grain slope. Linear expansion was also noted to increase with increasing slope of grain, whereas swelling and spring-back decreased.

Laufenberg (1984), extended the work of Lehmann et al (1974) when investigating the relative effects of forming pressure, resin content, and flake thickness on bond quality in phenol bonded particleboard by subjecting specimens that had a uniform density through their thickness to tensile loads applied parallel to the surface. Microscopic examination of the fractures induced revealed that both an increased resin content and the increased compaction obtained under higher press pressures, produced better bonding. It was concluded that the better dispersion attainable with a larger amount of adhesive combined with the increased compaction caused more flakes to be "spot welded" together. For example, an increase in adhesive content from 3% to 5%, enhanced strength by 20%, which was similar to results recorded by Geimer et al (1975), and an increase to 9%, added a further 15%. An increase in press pressure from 2.1*MPa* to 3.2*MPa* raised strength around 70%. Doubling the pressure to 4.2*MPa* doubled the strength. Flakes 0.5*mm* thick gave higher board strengths than either 0.25*mm* or 1.0*mm* flakes because fewer were broken by forming pressure. No significant difference in tension strength or modulus of elasticity was apparent with flakes between 0.25*mm* or 0.5*mm* in thickness.

Laufenberg (1984), also examined board strength in terms of the inclination of the wood fibre with respect to the direction of the principal stresses. Flakes with fibres inclined less than 5° provided 40% of the tensile strength, those inclined less than 10° gave 60%, and those below 30° totalled 85%. Consistently, most of the tension failures occurred in fibres inclined within 10° of the load direction, while most of the "un-failed" material was found to have a fibre angle less than 30°, reducing to no failures where the angle exceeded 50°. The relationship, for all practical purposes, was identical to the strength of solid wood with respect to the direction of the grain as expressed by Hankinson (1921).

Generally speaking, whether boards were homogenous or three layer, and whether loaded normal or parallel to the surface, the inter-relationship between strength and elasticity was shown to be consistent for any particular type of particleboard. Tensile strength perpendicular to the surface, known as the internal bond strength, was similarly consistent, and was shown to be a reliable indicator of strength and elasticity and, hence, board quality.

2.3.4 Adhesive Type

The economic balance between direct cost and its effect on processing cost influences the choice of adhesive. The waterproof and weather resistant resins are generally around three times the cost of urea-formaldehyde, but with curing periods some threefold longer, slow the process and reduce through-put. The curing periods of the tannin and phenolic resins have been halved over recent years, however, this reduction is accompanied by a proportionate increase in reactivity, and some consequential increase in difficulty with process control. But melamine fortification of urea-formaldehyde does not appreciably increase the initial cure period which remains well below that of tannin and phenolic formulations. While the unit cost of phenol-formaldehyde has reduced to become nominally the same as melamine-formaldehyde and melamine-urea-formaldehyde, the faster curing and better tolerance to the gluing variables of the latter lowers processing costs without impairing mechanical properties. Compared with phenolics, melamine-formaldehyde and melamine-urea-formaldehyde needed 30% less mixing water, and less again compared with tannin resins. Consequently, the delamination problems that attend the formation of steam bubbles during pressing are reduced with the lower water content of urea and melamine formaldehydes. They are thus easier to handle and result in a higher throughput due to their shorter press closure periods. Reduced press times also result in boards becoming appreciably denser towards the surface, thereby gaining in strength and stiffness. Cross-sectional density gradients were the same whether bound with urea or phenol-formaldehyde adhesives.

Obviously, the mat's moisture content is a prime determinant of strength and elasticity, and any reduction in the amount of initial mixing water required by the adhesive is beneficially reflected in better mechanical properties achieved more consistently. Some processing characteristics of various particleboard adhesives in current use are given in Table 2.1.

Table 2.1 Processing Characteristics of Particleboard Adhesives.

Adhesive	Mixture % solids/water	Cure (min) 20mm board	Chemical Reactivity	Relative Cost
Urea-formaldehyde	60/40	5	Low	1
Melamine-formaldehyde	70/30	5	Low	4
Melamine-urea-formaldehyde*	60/40	5	Low	3
Phenol-formaldehyde.	30/70	8	High-med.	4
Tannin-formaldehyde	40/60	4	V.high-med	3
Isocyanate		5	Low	3

* 60% melamine : 40% urea.

2.3.5 Process

Boards bound with formaldehyde resins were examined by Heebink et al (1972) for the effects of various factors on the rates of press closure and initial curing periods in the press. Their conclusion that the *"minimum press time could be determined as the point when the pressure required to hold the board to nominal thickness fell below the internal bond strength of the board"* indicated clearly that the rate of cure of the adhesive governed the production rate. To improve production so that more boards were pressed in each cycle, the options were to either reduce initial cure time, or increase the press size so that more boards were pressed in each cycle. The balance chosen between these would obviously affect the whole press line design and operation. The shortened press closing times obtained with lower moisture contents offered strong incentives to reduce the moisture in the particle mat through the use of drier wood and adhesives that required less water.

Boards made with isocyanate adhesives however, by virtue of their need for hydroxyl ions to effect polymerisation, exhibit a very much wider tolerance to moisture in the wood fibre from virtually zero to 25%*mc*. Hence, the more thorough drying of the wood fibre to the very low moisture contents around 4% needed with formaldehyde binders, is not nearly so important. The development of bonds strong enough to prevent delamination on opening the press when wood fibre moisture contents exceeded 20% lead to press cycles appreciably shorter than those attained with melamine and urea-formaldehyde, and around half the periods needed with phenolics. Relatively lower drying costs, reduced risk of fires in chip dryers, fewer emission problems and reduced steam formation, considerably lessen processing costs. Further reductions in costs also result from simpler glue preparation where the need for additives, such as hardeners, could be avoided and waste-water purification made less demanding. Isocyanates appear to hold significant processing advantages over formaldehydes.

2.3.6 Variability

The significant manufacturing variables affecting structural properties are; flake or fibre geometry, the ratio of glue solids to water, the amount of adhesive incorporated, wood fibre density and moisture content, the proportion of branch-wood, average board density, density gradients within the board, and wax content. The dominance of the first three enabled Hunt and Suddarth (1974) to develop a structural analogue for rapidly predicting properties based on particle shape and glue properties only, hence avoiding the need for much of the time consuming trials required to design a particleboard furnish to meet specified properties.

It is quite evident that the manufacturing process is intimately designed around wood fibre production methods and adhesive characteristics and that the limitations thereby imposed on processing lead to low product variability. Bryan (1960) found that particleboard produced in the USA in the early stages of development, exhibited variability exceeding 30%. McNatt (1973) studied a number of particleboards with differing species and flake shapes, and found that the largest coefficient of variation achievable in mechanical properties, 30% for modulus of rupture, was attributable to over-large flakes in thin board. More commonly, much lower coefficients of variation, between 8% and 16% for modulus of rupture, the level of which lay between 16*MPa* and 20*MPa*, were otherwise found in the range of boards McNatt examined. This work was generally supported by Lehmann and Geimer (1974), who also found that the widest possible range of manufacturing variables produced maximum coefficients of variation between 25% and 30%. Laufenberg (1984) examined the same effects in detail and recorded the same results.

With respect to directional effects, McNatt (1973) found that specimens cut both along and across the board showed a variation of around 10% within the average values, while for any particular direction coefficients of variation were as low as 4% or 5%. Part of the variability in McNatt's board was therefore attributable to the tendency of flakes to align to a nominal extent with the motion of the mat during its formation. Hunt (1976), putting under test material made in the United Kingdom c 1968, recorded that the short term strength of specimens cut across the forming direction was some 25% lower than specimens cut longitudinally. As confirmed by Adam (1997) and currently by the writer's testing, present day mat forming techniques eliminate directional effects imparting true isotropy in the plane of the board. Notwithstanding its directional variability, the standardised experimental board developed by McNatt could be considered close to the ideal structural board, and it generally exhibited better mechanical properties with lower coefficients of variation than most of the particleboards examined by other researchers at that time.

The standard of manufacturing, and hence mechanical properties, was appreciably lower in 1970 than it is currently. For example, in the United Kingdom the British Standards Institute prepared Draft Document 76/12154 DC in 1976, with coefficients of variation in strength up to 15%. Manufacturing improvement during the intervening 13year period is reflected in its issued form as BSS 5669-1989 which specifies a much superior structural board, identified as Type C5, that exhibits variability below 10%, property levels some 25% higher, and is virtually isotropic in-plane.

Hanley et al (1985) assessed all Australian manufactured particleboard and obtained for small standard test specimens, coefficients of variation between 7.7% and 13.6% for planar shear strength and between 4.8% and 14.3% for bending strength, and somewhat lower variations for moduli of rigidity and elasticity, the higher variability reflecting comparatively poor process control by some manufacturers. In the intervening twelve years, process control improved to the point where Adam (1997) sampled flooring grade particleboard made by the same manufacturers and measured coefficients of variation for bending strength and modulus of elasticity of around 10% for small standard test specimens and around 4% for full sheets. The difference suggests that standard test methods under-estimate true properties of large sheets by nominally 2%.

More recently, (1998), the writer measured similar coefficients of variation in small specimens of 19*mm* flooring grade board from a major Australian manufacturer confirming an improvement in 5%ile property values of around 25% since the assessment by Hanley et al (1985). This board no longer exhibited directional effects due to forming indicating the complementary development of planar isotropy. Another characterization of Australian particleboard by the writer c2000 that included planar properties, confirmed low variability for all mechanical properties, but some small difference in the level of properties was evident between boards from different platen levels within the press.

2.3.7 Discussion

The manufacturing process is primarily sensitive to two factors, the total amount of water contained in the adhesive and wood fibre that constitute the particle mat and the adhesive's chemical reactivity. Because the type of adhesive exerts a dominant impact on the board making process, any change in adhesive formulation brings in train modifications to the particle mat and the press cycle, possibly to the press itself and the whole processing line. By comparison, wood variables exert a relatively secondary affect on manufacturing. Consequently, the manufactured product exhibits a high tolerance to the inclusion of wood fibre from what may earlier have been regarded as waste. Whereas the incorporation of branch-wood affects dimensional stability appreciably and decayed wood lowers strength, the addition of plywood or particleboard waste was shown to have a marginal effect on mechanical properties.

The mixing of various species of softwood was found to have no effect, and forestry thinnings of mixed species of juvenile Australian hardwoods have been successfully made into particleboard, but no published work has been found on the effects of mixing softwood with hardwood. Species variation from mill to mill may occur to a limited degree in some areas of Australia, where, for example, both Radiata and Cypress pines could be mixed. But it would not be common and most mills would be designed to produce structural quality board wholly from either hardwoods or softwoods. It was noted that Hunt (1976) referred to boards termed as mixtures of hardwood and softwood, but no details were given.

It is apparent that any plant built to cope with the full range of all possible material and processing variables would become too complicated for commercial operation. Because these variables are closely linked to productivity, once a plant is in operation, there would need to be shown significant economies or market pressures to justify major modification with the attendant capital costs and loss of production. Manufacturing plants are designed very carefully to cope with the much narrower range of material variables inherent in the particular raw wood source on which the plant is based. Not unexpectedly, the final product exhibits much lower variability in mechanical properties of than most structural materials, which attaches a high level of confidence to its reliability. With variability in strength between 5% and 10%, particleboard is more similar to wood free of natural defects than to normal structural grade timber which commonly has a variability of in the order of 40%.

Heebink, Geimer, and Laufenberg, each demonstrated that, in the plane of the board, both tensile strength and tensile modulus of elasticity are related to the wood fibre angle, as distinct from the chip angle, in accordance with Hankinson (1921). In the absence of glue-to-wood bond failure, strength is limited by wood failure. When insufficiently bonded, flakes aligned with the load obviously contribute less to total board strength, because they break at their adhesive sites rather than through the wood fibre itself. The investigations suggest that optimum bonding is achieved when flake thickness is close to 0.5*mm*.

In a sheltered situation, the mechanical behaviour of particleboard is independent of the adhesive type except for the initial creep that develops when first exposed to ambient conditions. Urea bonded boards show the lowest creep, and phenolic bonded boards show creep some three fold greater. Dinwoodie et al (1984) noted that the creep of phenolic bonded boards is affected markedly by resin alkalinity and associated reactivity. The opinion expressed in the Technical Note, CSIRO (1962), that phenols were the "*most acceptable binders*" overlooks their greater creep, their increased chemical reactivity and associated increase in process control costs, and their higher first cost. From both a manufacturing and engineering design standpoint this is difficult to sustain for board applied in the normal protected building environment, where reliable structural performance does not depend on the use of a totally water proof adhesive.

To exclude the use of unfortified urea bonded board in a 'dry' environment on the grounds of structural performance appears unsupportable from the standpoint of mechanical behaviour alone. However, such exclusion would be viewed by engineers and building regulators as necessary to avoid failures due to hydrolysis brought about by excessive moisture contents attributable to poor building practice. The very high water resistance afforded by melamine fortified ureaformaldehydes, with initial creep rates half those of phenolics and enhanced structural reliability through their more amenable processing characteristics, offsets this objection and should place them in a preferred position as adhesives for structural particleboard. Despite this, phenolic adhesives are universally used for structural applications

The governing manufacturing variables affecting the structural quality of particleboard have been identified and are fully controlled by AS/NZS 1859.1-1997 which imposes lower limits on;

(a) board density and moduli of rupture and elasticity to control adhesive content, wood quality, flake geometry, flake dispersion, forming pressure and density gradients, and on:

(b) thickness stability and internal bond strength to govern adhesive content, adhesive dispersion, density, permeability and moisture resistance.

Relatively little work exists on the creep and relaxation of particleboard under either tension or compression in the plane of the board, most investigation being confined to loads applied normal to the surface. Mechanical behaviour when loaded in-plane is, however, linearly related to that when loaded normally.

2.3.8 Conclusions

The complex factors involved in the manufacture of particleboard are well understood and the resulting material is suitable for structural use. Exceptionally low variability of mechanical properties, not dissimilar to solid wood free of defects, renders particleboard manufactured to meet AS/NZS 1859.1-1997 for flooring applications, a highly reliable structural material when protected from excessive moisture absorption.

Strength and elasticity are related to the wood fibre angle, as distinct from the angle of the chip, with respect to the axis of the board and follow Hankinson's rule. Practically speaking, normal random particleboard as currently manufactured exhibits no orientation effects parallel to the surface and may be treated as isotropic in the plane of the board. The strength and elasticity of particleboard with a given density, are slightly greater than one half that measured along the grain of the parent wood with the same density and are independent of the mechanical properties of the adhesive.

When protected from excessive moisture, mechanical properties increase in value according to the amount of adhesive employed up to approximately 10%. Beyond 10% board strength is governed by wood fibre strength.

All mechanical properties are inter-related and remain constant for any given type of particleboard whether loaded perpendicular or parallel to the surface.

Within the normal range of density, between $650\,kg/m^3$ and $720\,kg/m^3$, the mechanical properties of normal random 3-layer particleboard are directly proportional to average density.

2.4 MECHANICAL BEHAVIOUR

2.4.1 Introduction

In this section of the literature review, the mechanical behaviour of particleboard and its relationship to wood in various environments is examined. The mechanical behaviour of wood and reconstituted wood panels is interrelated and involves complex physical interactions within the wood material under the influence of stress, moisture and heat. Fundamental rheology is yet to be explained, the extensive literature suggesting that long-term behaviour under load is governed by two distinct mechanisms; one, a time-dependent viscoelastic effect, the other, a moisture-dependent mechanosorptive effect. Both effects produce a continuing loss of strength and increase in deformation under sustained load until failure or, as suggested by recent literature, a stable limit state is reached.

Subject to loads for short periods, wood and reconstituted wood panels behave elastically, and long-term structural behaviour is estimated through factors applied to short-term elastic behaviour. The prime factors, k_1 in the resistance equation 1.1 and j, in the deformability equation 1.2, are duration-of-load factors that account for long-term effects, respectively, loss of strength and increase in deformation with time. It is necessary to view the values of k_1 currently specified in design codes in the light that their evaluation is the result of extrapolations of numerous studies of timber made over relatively short periods of one or two years. Similarly values of j are based on continuous creep monitoring of wood for 13 years, (Kingston 1968) and particleboard for 10 years (Dinwoodie et al 1992). In the absence of an explanation of fundamental rheology, the literature thus reflects considerable uncertainty; consequently, duration-of-load provisions currently made in structural design codes are in question.

The mechanical behaviour of particleboard may, in part, be inferred from its rheological similarity to solid wood and the other commonly used wood-based panels, plywood and hardboard. A study of the rheology of particleboard commenced with Bryan (1960). Investigations since have generally been limited to short periods with two exceptions; one, a continuous six-year study of stress relaxation, Hunt (1976), that commenced around 1968, the other, a five-year study of creep by Dinwoodie et al (1991). After 50 years under sustained load in protected external situation, Hunt estimated that around 0.20 of short term ultimate strength is retained and Dinwoodie et al, and that, deformation relative to elastic deformation increases to around 4.5 under a load of 0.24 short term ultimate strength.

Early investigations were generally confined to steady moisture conditions at room temperatures. Subsequently, investigation broadened to include the variable moisture and temperature conditions that significantly reduce strength and increase deformation. Unfortunately, in the endeavour to set behavioural boundaries, much of the work subjected the material to levels of stress well in excess of 40% of short-term ultimate strength and moisture contents as high as 20%, both now seen as much too high for structural use. With studies made under less extreme conditions, a better, but not yet complete, appreciation of the effects of ambient variations in moisture and temperature resulted. Relatively little work exists on creep and stress relaxation under either tension or compression in the plane of the board, most investigation being confined to loads applied normal to the surface. Fortunately, as earlier discussed, mechanical behaviour when loaded in-plane is linearly related to that when loaded normally.

2.4.2 Effect of Adhesive.

No significant work to establish the specific creep characteristics of adhesives commonly used for particleboard manufacture appears to have been published. However, there is general agreement in the available data that, following the first 600 hours under load, all particleboard exhibits similar creep characteristics irrespective of the adhesive. Limited work on the mechanical properties of particleboard adhesives is available. Simpson and Soper (1968), performed tensile tests on 'dumbbell' samples cut from thin sheets of a resorcinol-formaldehyde cured for periods up to 52 days and measured values of the modulus of elasticity in the order of 4000*MPa*. This is similar to that of particleboard itself whether bound with formaldehyde or isocyanate adhesives.

CSIRO (1970), referred to there being no appreciable difference between the creep behaviour of particleboards with similar furnish when subjected to cyclic moisture changes, whether bound with urea or phenol formaldehyde. Armstrong and Grossman (1971) observed that the difference in creep was due to the nature of the component bonding.

Gressel (1972) studied creep in wood, plywood and particleboard for the effects, in isolation, of moisture, temperature, adhesive type and bending stress, and found that creep rates were independent of the adhesive. Gressel concluded that the different creep rates observed depended on the differing rates of moisture adsorption that each type of adhesive characteristically imparts to the particleboard.

Halligan and Schniewind (1972), Haygreen et al (1975), found that phenolic and urea particleboards displayed similar long-term creep when placed under a range of humidity well in excess of that found in the normal building environment. The differences in initial creep rates are the result of a bonding breakdown identified by Christensen (1975). Hall et al (1977) demonstrated that, after the first 600 hours, subsequent creep rates of all reconstituted wood panels are virtually identical, strengthening evidence that the type of adhesive has no recognizable effect on long-term creep rates. Clad and Schmidt-Hellerau (1981) found that in an ambient external protected situation, particleboard bound with phenol-formaldehyde developed an initial relative creep nominally twice that of board bound with melamine-formaldehyde and nearly thrice that when made with urea-formaldehyde.

2.4.3 Effect Of Forming Stresses

Christensen (1975), in referring to initial creep, regarded the surface of the chip as being partly composed of damaged fibres weakly connected to the chip, so that even low levels of stress could readily break both weak fibres and weak adhesive bonds at their various adhesion sites. Covering part of the chip surface with wax and other surface containments would also prevent adhesion locally and create stress concentrations in adjacent bonding sites. When moisture is first adsorbed in bringing the board's low moisture content and high temperature on leaving the press into equilibrium with ambient conditions, some of the considerable strain energy, stored in the board as a consequence of the work done by the compressive forming force impressed during processing, would be released by breaking weak adhesion sites. A relatively larger number of breakages would also occur when a board was first subjected to externally applied load, particularly if accompanied by internal stresses induced by moisture change.

It is apparent that the creep that results initially would be irrecoverable and magnified disproportionately by comparison with subsequent creep and is explained as substantially due to a progressive release of locked-in forming stresses over the first 600 hours or so under load. Halligan and Schniewind (1972) and Geimer et al (1973) found that steam curing was an effective method of removing high initial creep.

2.4.4 Effect of Stress Fatigue

McNatt and Werrin (1975) examined the fatigue strength of three types of random particleboard, two were normal three-layer boards, one bonded with a phenolic resin the other with a urea resin, while the third was a single-layer urea bonded board. Tests were conducted at 50%*rh* (6.5%*mc*) and 25°*C*, and after 10 million cycles residual strength was about 45% of short-term ultimate strength irrespective of whether the load applied was parallel or perpendicular to the surface. The minimum stress level was 10% of the short-term tensile ultimate strength, the maximum 30%. The data indicated that the number of cycles to failure would not alter significantly with any other range of stress within the same limits. Both initial strength and residual strength after 10 million cycles were independent of either the glue type and average density. The residual strength of the two three-layer boards was similar, but only about half that of the denser homogeneous board, the fatigue strength of which was similar to that reported for hardboard.

Van der Put (1989) examined the fatigue strength of particleboard over a wide range of frequencies and concluded that the loss of strength could probably be explained by a unique dominant mechanism that coupled long-term strength to fatigue strength. The mechanism was not discussed.

2.4.5 Effect of Heat

2.4.5.1 Effect of Heat on Wood

The Wood Handbook (1974) referred to a linear inverse relationship between strength and temperature that, for wood, was considered reversible up to $100°C$. Irrespective of the level of stress, reduction in strength and increase in deformation is linear with increasing temperature and vice versa. But prolonged exposure to temperatures above $200°C$ damages wood material permanently. Prolonged exposure of wood with moisture contents around 10% to temperatures well below $-50°C$ increases mechanical properties. In the absence of literature specific to particleboard, it is reasonable to expect that its behaviour is similar.

Hearmon and Paton (1964) concluded notionally that the creep of wood increased but slightly with an increase in temperature, their experimental work offering no reliable estimate of the actual creep rate. Armstrong (1966) reported that creep in seasoned wood in bending at $40°C$ was approximately double that at $25°C$. Brock (1973) examined the bending strength of wood cycled daily from 20°*C* to 40°*C* and 20°*C* to 60°*C* for 2 years. No significant change in either strength or elasticity was observed, but a slight reduction in the work expended to failure suggested some small embrittlement of the wood. Arima and Grossman (1978) noted that wood subjected to a rise in temperature of 80°*C* during drying in a deformed state lost some of its capacity to recover. They considered that the new, stable, bonding configurations develop a "permanent set" upon cooling which may be largely relieved by subsequent absorption of moisture.

To investigate the effect of cyclic temperature on stress relaxation, Fridley et al (1989) subjected Select and No. 2 Structural grade timber to a 15%ile bending stress and 12hourly cycles of temperature between 23°*C* and 38°*C* for 7 weeks during which the test chamber was maintained at 50%*rh*. Similar initial moisture contents averaging 9.7% (select) to 9.9% (No. 2 grade) were recorded and it was noted that "*little change*" occurred between start-up and failure and that therefore "*no hygroscopic effects were assumed to be present*". Constants derived from earlier tests at constant temperature and under the same stress levels were used for the damage accumulation model that was fitted to the cyclic data. The model assumes that stress relaxation is linearly related to temperature and the logarithm of time-to-failure effect. Most of the failures occurred during an increase in temperature or while at the high temperature part of the cycle and were attributed to the influence of temperature. Stress relaxation due to mechanosorptive action was not considered.

Le Van et al (1990) concluded that "*The lack of a reduction in MOE, MOR, or WML (work to maximum load) ------ at 89°F (27°C) and 130°F (54°C) suggests that no permanent thermal degradation in strength occurs in exposures at 130°F (54°C) for up to 6 months.*

Although the National Design Specification for Wood Construction (Section 2.2.2 and Appendix C, NFPA 1986) allows intermittent exposure to up to 150°F (66°C), our findings support the idea that thermal effects are immediate and recoverable in nature for exposures at 130°F (54°C) for up to 6 months."

Moore (1993) continued the work of Brock (1973) by subjecting wood in bending to temperature cycled daily between 20°*C* and 90°*C*. After 3 years, a 70% to 80% loss in bending strength occurred that was near linear with temperature, but the modulus of elasticity remained practically unchanged. Moore concluded that the greater loss of strength under the cyclical regime was difficult to quantify in practical terms and therefore could not be compared with the loss under continuous heating. The slow cooling during the cyclical regime resulted in longer overall exposure to higher temperatures making it unclear whether the degradation was due to temperature cycling or simply longer exposure to heat.

2.4.5.2 Effect of Heat on Reconstituted Wood Panels

Haygreen and Sauer (1969) investigated the effect of temperature on the bending strength of wet-process hardboard held at a moisture content of 10%. It was shown that irrespective of whether the stress was 30% or 50% of the short term ultimate value, a rise in temperature from 20°*C* to 37°*C* reduced strength by 20% and increased deflection by 10%. Raising the temperature to 60°*C* reduced strength by about two thirds and nominally doubled deflection at failure. It was concluded that loss of strength and increased deformation were probably linear with temperature. Yang and Haygreen (1971) extended this work to particleboard, found that an increase in temperature from 20°*C* to 30°*C* induced around 50% more deflection, and reached the same conclusion. They regarded the similar behaviour of particleboard and hardboard as further confirmation of the rheological similarity noted earlier by Sauer and Haygreen (1968).

Dinwoodie et al (1991) subjected a range of particleboard types to 0.30 of short-term ultimate bending strength in a sheltered outdoor environment for 5 years. It was concluded that changes in ambient temperature, which varied from -8°*C* to 22°*C*, had *"little or no effect on creep"*.

2.4.6 Mechanical Behaviour in Steady Moisture Conditions.

2.4.6.1 Creep of Wood and Reconstituted Wood Panels

Bryan (1960) recorded that, in bending with the load applied normal to the surface, the relative creep of particleboard was nominally double that of plywood. Whether the relative humidity was 50% or 80%, relative creep remained in the ratios, 3.3 for particleboard to 1.6 for plywood and was thus independent of moisture content when held steady. The relative humidity respectively corresponded to moisture contents of 9% and 16% in the plywood, and 6.5% and 12.5% in the particleboard. Bryan and Schniewind (1965) subjected particleboard and plywood to 0.30 of short-term ultimate strength in bending at 25°*C* and 65%*rh*, (8%*mc)*, and measured a relative creep over 3 months of 1.5 for particleboard, which was double that of plywood similarly loaded for the same period. Halligan (1965) held two groups of particleboard specimens in bending for three months at 40°*C*, one at 30%*rh* the other at 70%*rh*, corresponding to 5.5%*mc* and 8.5%*mc* respectively. The relative creep measured respectively 3 and 3.5, the latter still rising slowly. Plywood, similarly tested, showed a relative creep of 2.8 in both environments.

Perkitny and Perkitny (1966) loaded wood, particleboard and hardboard in bending at 20% and 40% of short-term ultimate strength and measured creep for ten days at constant moisture contents of zero, 10% and 20%. At any particular moisture content, relative creep was found to be directly proportional to stress and nominally in the ratio of 1:2-3:4:5 for wood, plywood, particleboard, and hardboard respectively. The plywood ratio of two was obtained with face plys parallel to the span and three when plies were perpendicular. Relative creep increased as moisture content increased. Yang and Haygreen (1971) also showed that the relative creep of particleboard increased with higher moisture contents and that the level of stress had a negligible effect. The ability of their method to predict deflection was appropriate at 50%*rh* (6.5%*mc*), but had little validity beyond 80%*rh* (12.5%*mc*) when strength began to decrease rapidly.

Armstrong and Grossman (1972) held particleboard at 35°*C* for 40 days under a bending stress of about 15% of short term ultimate strength and recorded relative creep of 1.5 at 6%*mc* and 4.5 at 18%*mc*, respectively 40%*rh* and 95%*rh*. They also confirmed the finding by Perkitny and Perkitny (1966) that creep of hardboard was some 25% greater than particleboard. McNatt (1974) and Halligan and Schniewind (1974), also found that the mechanical properties of particleboard changed little in environments with constant humidity, whether 30%*rh* or 70%*rh*, inducing respectively 5.5%*mc* or 9.5%*mc*; but reductions began to occur much more rapidly beyond 75% or 80%*rh*, i.e., 11 or 12%*mc*.

Haygreen et al (1975) compared the creep of plywood, particleboard, and a form of particleboard made with aligned flakes, known as oriented strand board, (OSB), subjected to bending. When held at 21°*C* in constant relative humidity of 65%, 70%, 75% and 80%

(moisture contents of 8%, 9.5%, 11% and 12.5%.), a marked increase was observed beyond 75%*rh* for all materials; the creep of OSB lying between the lower value for plywood and the higher value for particleboard. The relative creep of particleboard was found to be about the same as plywood around 65%*rh* (9.5%*mc*), but double at 85%*rh* (14.5%*mc*), which was similar to that noted by Bryan and Schniewind (1965). The foregoing confirmed the earlier conclusions of Bryan (1960), Bryan and Schniewind (1965) and Yang and Haygreen (1971), that at any given moisture content below 12%, relative creep, while proportional to stress, did not vary greatly with a level of stress below 40% of short-term ultimate strength, but commenced to rise rapidly when moisture contents exceeded 12%.

Dinwoodie et al (1991) observed that under the same stress, moisture and temperature regime, "*the ratio between the average relative creep at all times periods of solid timber to all chipboards was approximately 1:2, less than the 1:4 found by Perkitny and Perkitny (1966)*".

2.4.6.2 Stress Relaxation of Wood and Reconstituted Wood Panels

Bryan (1960) held particleboard bonded with an adhesive content of 11%, (6.5% urea-formaldehyde, 4.5% phenol-formaldehyde), for up to 1000 hours under three point bending with relative humidity constant at 30%, (5.5%*mc*). Time to failure was measured for levels of stress ranging from 0.6 to 0.9 of short-term ultimate strength. By extrapolating a straight line regression for stress ratio against natural log-time, the residual stress ratio after 27 years was estimated to lie between 0.56 and 0.59 of the short term ultimate stress. Bryan also noted that the deflection curve was characteristically similar to the Madison curve (Wood 1951), except that deformations were greater. Relaxation and creep were considered to be independent of board density. From a two-year study, Kufner (1970) predicted that the residual bending strength of particleboard after ten years under load would be around 0.67 with 10% moisture content and 0.33 with 20% moisture content, as acquired in reative humidity of 72% and 95% respectively. Again, determined by extrapolating a straight-line regression of relaxation with log-time.

McNatt (1975) studied the effects of the time taken to apply the full test load to particleboard specimens loaded in bending normal to the surface. It was observed that, for each tenfold increase in the period to full test load, bending, tensile and shear strengths decreased close to 8% on average. Similar results were registered for tensile loads applied parallel to the board surface. McNatt noted that the effect, which was virtually the same as that reported for hardboard and wood when similarly loaded, indicated a close rheological relationship between all three materials.

2.4.7 Mechanical Behaviour In Fluctuating Moisture Conditions

2.4.7.1 Creep of Wood

Armstrong and Kingston (1960, 1962), Armstrong and Christensen (1961), Christensen (1962) and others, hypothesised that the creep of wood under load is induced not only by viscoelastic action, but also by mechanosorption, a phenomenon caused by the migration of water molecules through the cell walls. Wood cell walls are composed of chains of cellulose molecules, part crystalline and part amorphous, with water bound to the surface of the cell lamella by capillary forces. In passing through the cell wall, water is thought to cause differential swelling and shrinking of adjoining layers of the lamella, which under the reactive internal forces set up to resist externally applied forces, induce shearing forces between them. Compressive components of these internal forces produce local buckling in some of the cell walls, while their tensile components cause portions of the curvilinear crystalline molecular chains to straighten permanently. The shearing forces progressively break various hydrogen bonds causing slippage between molecules. The creep induced by moisture changes while subjected to externally imposed loads is a continuation of these internal actions. When moisture movement ceases, molecular bonds reform in new displaced locations, slippage ceases and behaviour became purely elastic once more. Irreversible, cumulative extensions are the result; the greater the original moisture change the greater the deformation.

The observed deformation accompanying desorption appeared much larger than the partial recovery that occurs with subsequent adsorption, but the physical action is probably the same. Christensen (1962) considered the partial recovery of the "*frozen-in*" creep deformation to result from a relocation of matrix material when taken through another moisture cycle following removal of the external load. This relocation is caused by desorption which reduces the volume of matrix material thereby releasing the locked-in strain energy of the constraining compressive forces. Christensen (1962) also observed that mechanosorptive deformation is independent of the level of applied stress.

Armstrong and Kingston (1962) showed that mechanosorptive creep depended on the magnitude of moisture change, not on the rate of change and that most of the creep is recoverable when moisture changes are cycled. Hearmon and Paton (1964) referred to this earlier work and made additional observations concerning the effects of humidity on wood.

When relative humidity decreased from 90% to 30%, that is, as moisture content reduced from the "green" to the "dry or seasoned" condition, it was noted that the accompanying nominal 25% increase in Young's Modulus was consistent with the increase accompanying a reduction in moisture content from 20% to 6%. But with successive cycles of relative humidity from 90% to 30% returning to 90%, (green/dry/green), no change in Young's Modulus could be observed. Their work also showed that under a load of 37.5% of short-term ultimate bending strength, failure occurred during the 14th cycle accompanied by a relative creep around 25. However, after 10 or so cycles at a 12.5% load level, the relative creep curve virtually asymptoted to a value of roughly six. By comparison, under the same load, but under steady 93%*rh*, (18%*mc*), relative creep nominally doubled before levelling off.

Armstrong (1966) summarised several years of investigation into the creep of wood in bending under ambient conditions. When loaded in bending to 40% of short-term ultimate strength, deformation nominally doubled during the first year. Subsequently, deformation remained virtually constant during winter increasing during summer only. Kingston (1967) reported that under compression parallel to the grain, non-recoverable creep increased steeply when stress exceeded 70% of the short-term ultimate. Kingston (1968) observed that structural timber scantling situated outdoors creeps at nominally double the rate when situated indoors. Continuing this investigation, Kingston (1968) examined the influence of the magnitude of bending stress on the deformation while drying of structural grade scantling and concluded that up to 35% of short-term ultimate stress, compression parallel to the grain was elastic. Kingston (1969) then examined pairs of beams with eccentric neutral planes arranged so that one of each pair had a high stress on the compression side, the other an identical high stress on the tension side. Beams with high compressive stress deformed more during reductions in moisture content than their counterparts with high tensile stress. Further work confirmed that changes in deformation due to moisture variations were greater under compression than under tension. Kingston (1968) also reported on some structural grade scantling that, by then, had been subjected to bending under dead load for 13 years in Melbourne. Under indoor conditions, an annual increase in deflection of some 4% of the initial elastic deformation was observed. In an unsheltered outdoor situation the rate practically doubled to around 7%. Kingston believed that after this lengthy period the *"Time dependent viscoelastic creep has long since ceased to contribute to the deflections but moisture loss during each summer continues to cause increases in deflection."*

Armstrong (1972) demonstrated that the strain accompanying a constant compressive stress increased with a change in the distribution of moisture through the specimen, but was unaffected while the distribution remained uniform. Armstrong believed *"that the transient effects of load and moisture content change in wood may be explained in terms of changes in the macromolecular configurations in the amorphous regions of the cell wall, but only if volume changes occur in the wood substance during water movement. Such volume changes during sorption of water could give rise to changes in the hydrogen bond configurations and under the bias of an external force additional relative displacements may occur between cellulose chains in the wood substance".*

Pure crystalline cellulose chains are impermeable, (Preston 1974), therefore, their hydrogen bonding is unaffected by either the moisture content of the wood material or moisture movement through the wood material.

When examining deformation in collapsible and non-collapsible species of Australian hardwood, Armstrong (1983) observed mechanosorptive phenomena *"whenever collapse of the cell wall occurred during the loss of free water from the cell lumens and also when normal shrinkage occurred with loss of chemically bound water from the cell wall" (but) " quantitative evidence is needed to show that distortions in the cell wall which accompany collapse ..... cause volume changes in the wood substance (and) no such evidence is yet available."* Armstrong (1983) demonstrated that structural timber with a moisture content around 10%, responds to ambient humidity fluctuations in a sheltered external environment with moisture content changes between 2% and 4% that dampen to between 1% and 2% in an internal environment. Because it is proportional to moisture change, the mechanosorptive creep of timber in sheltered external situations is thus expected to halve when situated indoors.

This leads to the conclusion that increases in the deformation of timber members under dead load due to ambient moisture changes are estimated by multiplying the initial dead load elastic deflection by;

1.0 when moisture content remains below 10%,

1.3 when moisture content is between 10% and 15%,

2.0 when moisture content is between 15% and 20%,

3.0 when moisture content exceeds 20%.

AS 1720.1-1997 specifies for structural grade timber more conservative values of long-term deformation than those above as respectively 2.0, 2.5, 3.0 under compressive stress, which values are halved for tensile stress. Madsen (1992) dealt subjectively with the effect of climate on the creep of timber after 30 years as shown in Table 2.2, but without explanation. The tabulated values show that the recovery in deformation occurring between successive applications of live load may be accounted for by adding nominally 10% of live load deflection to long-term dead load deflection. To account for mechanosorptive effects, long-term dead load deflection in a nominally steady environment is increased approximately 50%.

Table 2.2 Deformation Factors Proportioned to Initial Elastic Deformation for Structural Timber Members under Dead Load and Live Load for 30 years in Sheltered Outdoor Situations. (After Madsen 1992).

Type of Member	Deformation Factor		
	Steady Climate	Varying Climate	Varying Climate
Timber Beam	$2.5\Delta_{DL}$	$4.0\Delta_{DL}$	$0.2\Delta_{LL}$
Glulam Beam	$2.0\Delta_{DL}$	$3.0\Delta_{DL}$	$0.2\Delta_{LL}$
Truss Mech. Jointed	$4.0\Delta_{DL}$	$6.0\Delta_{DL}$	$0.2\Delta_{LL}$

Δ_{DL} elastic deformation when dead load is first applied.

Van der Put (1989) regarded the creep of wood to be influenced under compression by the amorphous lignin polymers, and in tension, by the crystalline cellulose material. Robson and Higgins (1989) considered creep deformation as most likely to occur at secondary bonds in the amorphous and non-crystalline regions of the wood cell wall and be caused by the displacement of bonds within the cell wall from one energy well to an adjacent well. Using a spring and dashpot rheological model due to Kingston and Clarke (1961), Robson (1988) expressed the creep rate in terms of the internal energy of wood under the action of imposed stress. While the model reproduces observed steady state creep phenomena satisfactorily, Robson and Higgins (1989) acknowledge that two effects associated with mechanosorptive action are not reproduced; the accelerated deformation that occurs during rises in temperature and the recovery of deformation during sorption of water.

2.4.7.2 Creep of Reconstituted Wood Panels

Bryan and Schniewind (1965) loaded particleboard at 25°*C* to 30% of short-term ultimate bending stress and after 42 days measured a relative creep of 3.5 after cycling moisture content between 6.5% and 9.5%, i.e., between 50% and 70%*rh*. Halligan (1965) used the same stress level at 21°*C*, increased the cycle to range between 50% and 80%*rh*, i.e. 6.5% to 12.5%*mc*, and obtained a similar relative creep of 3.3, but after a much longer period of nine months. He simultaneously measured a relative creep of 1.6 for plywood, and concluded that particleboard deformed nominally twice as much as plywood under the same stress in similar fluctuating humidity above 50%*rh* (6.5%*mc*). Bryan (1960) had reached the same conclusion when he tested both under steady conditions.

Sauer and Haygreen (1968) found that the deflection of "wet-process" hardboard loaded in bending to 30% of short term ultimate stress was increased by adsorption only, as obtained by raising temperature from 25°*C* to 37°*C*. Raising moisture content from 5% to 10% increased deflection between 15% and 20%, but under desorption and steady moisture conditions, deflection remained practically static. An increase in bending stress from 30% to 50% of short term ultimate stress nearly doubled the deflection, but the moisture-dependent deformation relationship remained virtually unaltered. In line with Armstrong and Christensen (1962), it was hypothesised that stress relaxation and creep was caused by shearing slippage created by broken hydrogen bonds and took place when water molecules passed through the wood material. Larger stresses increased the rate of slippage by breaking more bonds, as did higher temperatures. Creep recovery was suggested to result from restoring forces, which had been temporarily locked into the wood cell structure by the pressure exerted during manufacture, being relieved when hydrogen bonds that were originally broken by the passage of water, were re-established by further water movement. It was also concluded that particleboard and hardboard are rheologically similar.

Armstrong and Grossman (1972) found that the relative creep of particleboard under a bending stress of 0.15 of short term ultimate at 25°C lay between 9 and 11 when moisture content was cycled weekly between 6% and 18%, that is, between equivalent relative humidity of 40% and 90%. Held under the same stress at 35°C for 40 days, with steady moisture contents of 6% and 18%, relative creep was observed to be significantly less at 1.5 and 4.5 respectively. Desorption cycles were shown to produce more creep than adsorption cycles which conflicted with observations made by Sauer and Haygreen (1968). They also noted that cycling moisture content between 6% and 18% did not change Young's Modulus, the same finding as Hearmon and Paton (1964) with respect to wood. Armstrong and Grossman (1972) concluded that, in view of the qualitatively similar behaviour of particleboard, hardboard and wood during sorption of moisture, "*the basic mechanism .... is similar in the three materials and depends on features common to all, but not on the type of* *bonding of the components. On the other hand, the quantitative differences .... might be explained in terms of the nature of the bonding."*

Haygreen et al (1975) found that when both materials are held at 21°*C* and humidity is cycled daily between 50%*rh* and 70%*rh*, (6.5%*mc* and 9.5%*mc*), the creep of particleboard is double that of plywood. And when cycled between 50%*rh* and 85%*rh*, (6.5%*mc* and 14.5%*mc*), is thrice that of plywood. These results were practically identical with those reported by Halligan (1965) under the same temperature / cyclic humidity regime. In attempting to relate the variations that occurred under ambient conditions to those under steady moisture conditions, Haygreen et al (1975) concluded that *"boards subjected to the 55-65%rh cycles exhibited about the same creep as boards at a constant 60%rh, (and) the 50-70%rh cycles produced only slightly greater creep. Boards subjected to 40-80%rh however developed much more creep."* Similarly, Hall et al (1977) found that a cyclic 45% to 65%*rh*, (6% to 8.5%*mc*), caused about the same amount of creep in particleboard as the ambient interior environment, but somewhat more creep than occurred in steady 50%*rh*, (6.5%*mc*) conditions. This agreed with the general view that creep increased markedly when humidity was cycled between 45% and 85%*rh*, (6% to 14.5%*mc*), a range not found in natural ambient conditions in Australia. It was also noted that the difference in total creep between any of the reconstituted wood panels, including plywood, depended on their respective creep-rates during the first 600 hours or so, after which, under the same stress and environmental conditions, the rate reduced to one that was common to all panels.

Brynildsen et al (1976) referred to long-term creep factors of 2.5 for particleboard and 3 for hardboard in situated indoors and warned against use where moisture and temperature were cycled and in climates where large variations in moisture content were anticipated. Pearson (1977), in recommendations to the *National Particleboard Association* (USA), proposed a factor of 2 for relative creep under permanent load in internal environment.

Eurocode 5 specifies creep factors under permanent load when indoors or in a protected outdoor situation respectively as 2.5 and 3.3 for particleboard, 3.3 and 5.0 for hardboard, and 1.7 in both service classes for timber. Based on co-operative research by the *European Federation of Associations of Particleboard Manufacturers,* FESYP, c1980, the design standard, BSS 5268, Part 2-1996, specifies long term creep in terms of the ratio between computed "actual" stress and allowable stress. Under a design load equal to the maximum sustained load allowable, this results in a creep factor of 5.63 for particleboard and a similarly derived factor of 4.52 for hardboard. For both materials the factors apply to use in "dry exposure conditions" only.

Halligan and Taylor (1987), in preparing safe load tables for particleboard flooring, adopted a relative creep factor of 2 for a protected environment and warned that it could be greater in highly humid situations. Dinwoodie et al (1991) held a range of commercial particleboards in a sheltered outdoor situation for 5 years under bending loading of 0.3 of short-term ultimate strength. The creep rates of all boards bound with melamine-urea formaldehyde, subsequent to the initial period of nominally 600 hours, were similar and uniform. The rate after 5 years in ambient conditions was similar to that observed after 6 months under the same stress at $20°C$ and $90\%rh$, $(19\%mc)$.

2.4.7.3 Stress Relaxation in Wood.

The first scientifically based prediction of the loss of strength exhibited by wood under sustained load is due to Wood (1951) who predicted that 9/16 of short term ultimate strength is retained after 50 years in a sheltered external environment. The value was obtained by extrapolating a logarithmic expression, now known as the Madison curve. The Madison curve still universally underpins structural timber design, but, increasingly, 9/16 is being questioned as too high. Hunt (1976), Madsen (1992) and others have stated that the Madison curve is unsafe for structural design purposes. Armstrong (1966) predicted that, eventually, the long-term strength of timber would be shown to fall below half its short-term ultimate strength.

Since the initial attempt by Wood (1951), the duration-of-load effect on the strength of timber scantling has been variously modelled. Madsen and Johns (1982), Foschi and Barrett (1982), Gerhards and Link (1987) and others predict that retained strength approaches 0.50 after 50 years. Nielsen (1992) forecast retentions for No.2 structural grade scantling under a sustained (normal design) stress of 10.1MPa of around 0.57, and for glulam under sustained 34.5MPa, around 0.40. These latter researchers regard wood as a "*damaged viscoelastic material*" and base their forecasts on fracture mechanics and damage accumulation modelling. To avoid the need for experiments stretching to many years, model parameters are obtained from short-term data collected over periods ranging between a few months and one or two years. Even though the models fit fairly well the short-term data, all extrapolations seem to aim at some arbitrary residual strength in the vicinity of 0.50 after 50 years.

Grossman and Kingston (1954, 1963), Grossman (1971) demonstrated the matching form of the rate curves for stress relaxation and creep during a reduction in moisture content, over a period three weeks, from around 30% to 10%, that is, from "green" to "dry". Referring to the mathematical model for mechanosorptive deformation proposed by Leicester (1971), Grossman (1971) concluded that an improved agreement between calculated and measured relaxation could be obtained by adding viscoelastic elements to the mechanosorptive model.

Based on the literature, with no experimental verification, Boyd (1982) offered an explanation of mechanosorptive creep similar to Armstrong (1972, 1983). Under tensile stress, the cell volume is reduced so that successive moisture cycles densify the lignin matrix, reducing its capacity to absorb water and also causing some strain hardening of the cellulose material. The result is an increasing resistance to further deformation with the creep rate continuously reducing over time. Compression stresses expand cells laterally, shorten their length and create instability that results in local buckling in their walls. With the passage of time, continuing instability reduces cell strength well below short-term capacity. When subjected to bending stress, this instability together with matrix plasticity, allows shear displacements to occur between adjacent cells which progressively shift the neutral axis towards the tension edge to maintain the balance between the internal resisting couple and the applied bending moment. Boyd referred to Kingston and Armstrong (1951) who reported that "*a considerably greater change in length was found to occur in the compression than in the tension face*" of timber beams in a ratio between 3:1 and 4:1.

Fridley et al (1992) studied the mechanosorptive effect with scantling sized timber at 23°*C* subjected to a constant bending stress and cycles of humidity between 35%*rh* and 95%*rh* for 24 and 96 hour periods. Adopting the approach of several other researchers, among them Barrett and Foschi (1978a) and Gerhards and Link (1987), a damage accumulation model was developed which incorporated viscoelastic and mechanosorptive effects. Model parameters were fitted to provide a notional 50% residual strength after 50 years. It was concluded that *"the mechano-sorptive effects commonly observed in creep tests ----- are also present in the load-duration behaviour," and "the inter-dependence of creep and creep-rupture (load-duration) is quite evident with the observation of mechano-sorptive effects in load duration."* Neither an explanation of the phenomena nor a reason for the residual strength selected was advanced. From a study of structural grade timber under load for 12 years, Gerhards (2000), proposed a strength loss of 50% for a 10year load duration.

Ranta-Maunus (1999) examined tension perpendicular to the grain in 100*mm* and 140*mm* wide curved and tapered beams held under constant load and subjected to relative humidity cycles of 40% to 85% and 55% to 90%. From linear log-time extrapolation, it was concluded that the mechanosorptive effect nominally halved the time to failure, the larger sections being less sensitive to the cycling. It was also concluded that under constant humidity the duration of load effect at one year was slightly less than the Madison curve indicated. An impervious coating was shown to significantly mitigate the mechanosorptive effect by reducing induced stresses by around 70%.

Milner et al (1999) subjected specimens of clear wood nominally 1*mm* thick, 31*mm* wide and 200*mm* long, at 38°C, to compression parallel to the grain while humidity was cycled between 25%*rh* and 90%*rh*, (7%*mc* and 22%*mc*), for cycles of 15, 150, and 710minute duration. Under a stress of 10.4*MPa*, creep strain appeared to asymptote at roughly 750 μ after some 200 cycles or so of 15 minutes duration accompanied by uniform strain amplitudes around 150 μ. When stress was nominally doubled to 18.2*MPa*, creep, accompanied by uniform amplitudes close to 310 μ, recommenced immediately and was still rising after a further 10 cycles. Under 18.2*MPa*, both the 150 and 710minute cycles appeared to induce similar strain amplitudes. Echoing Toratti (1992), Milner et al (1999) state that "*no firm conclusion can be drawn about the existence or otherwise of a mechanosorptive limit. While a limit is apparent at 10.4MPa no such behaviour was observed at 18.2MPa*".

2.4.7.4 Stress Relaxation of Reconstituted Wood Panels

In unpublished work, Kloot c1960 predicted that hardboard under sustained load in a sheltered exterior environment retained around 0.24 of ultimate short-term strength in the long term. During 1970, Hunt (1976) commenced a six-year investigation into stress relaxation in urea-bonded particleboard. Three particleboards with different densities were placed in two ambient environments; a protected exterior environment and an interior environment. Levels of stress ranged from 0.2 to 0.6 of short-term ultimate strength, and thus extended observations by Bryan (1960). Hunt combined Bryan's observations for particleboard with a mean density of 720kg/m^3 with his own, and using a linear log-time extrapolation similar to Bryan's, confirmed Bryan's estimate of residual strength as 0.59 of short-term ultimate strength after 27 years in a sheltered external.

However, Hunt noted that "*linear .... extrapolation would .... provide unsafe results*", and reinterpreted Bryan's data using a cubic log-time expression to extrapolate a lower residual strength of 0.49 after 27 years indicating that sustained stress for 50 years would be limited to around 0.20 of short-term ultimate strength. Hunt's data indicates a residual strength of 0.17 for 680*kg/m*^{3} board. Both these boards were made with a nominal 1% wax content, and contrasted with 600*kg/m*^{3} board containing no water repellent that was estimated to have a 50 year strength limit of 0.13. This board would fail the V313 test specified in AS/NZS 4226-1995 and is of interest only because it demonstrates the much larger loss of strength attributable to excessive permeability in the absence of a water repellent.

Simultaneously, parallel tests were run on specimens held indoors for two years. Stress relaxation reduced appreciably to indicate a much longer life, but no attempt was made to quantify the shift. The data indicate that 50year residual strength in an indoor situation is greater by nominally 25% than that of particleboard in a protected exterior situation. Hunt's data suggests that 720*kg/m*^{3} board under permanent load in an internal environment, where relative humidity rarely exceeds 56%, retains around 0.24 of its short-term ultimate strength after 50 years. During Hunt's work, relative humidity ranged between 44% and 90% externally and between 40% and 56% internally, developing ranges of moisture content between 6.5% and 17.5% externally, 6% and 8% internally.

The Nordic Committee on Building Regulations (1973) expected particleboard under sustained load to retain 0.40 of its short-term strength in an internal situation and 0.25 in a protected external situation. McNatt (1975) and others concluded from linear log-time extrapolations of two or so years of observation, that particleboard in internal situations retained around 0.46 after 10 years under load. Following which period, according to the

American *National Design Specification*, (NDS), a further reduction in strength of 10% will occur, thus decreasing retained strength to around 0.40 after 50 years.

Pearson (1977) estimated that particleboard with similar density and mechanical properties would have a residual strength of 0.41 when held under a steady, but higher humidity that gave a moisture content around 11%, 75%*rh*. Pearson's projection was linear with log-time from 1000 hours of observation and derived by methods that are used to assess working stresses in defect free wood specimens. Pearson referred to the empirical equations developed by Halligan and Schniewind (1972) and Hunt (1976) for factors to adjust estimated long duration stresses for environmental moisture effects.

Investigations by Hunt (1976), subsequent characterization by Hanley et al (1985) and work by McNatt (1986), confirm that a linear relationship exists between strength and density, bringing into question Bryan's earlier (1960) conclusion that they are unrelated.

2.4.8 Summary of Rheological Effects

As drawn from the literature, matters that appear to influence the mechanical response of timber and particleboard to viscoelastic and mechanosorptive actions may be summarized as follows. The summary is confined to material, with average maximum moisture contents less than 15% in wood and 12% in particleboard, that is stressed below 40% of short-term ultimate strength.

1. Viscoelastic and mechanosorptive effects occur in wood and particleboard whether loaded parallel or perpendicular to the wood fibre, in tension, compression or shear. Both effects cause creep deformation and stress relaxation.
2. All mechanical properties of particleboard, in-plane and normal to the surface, are proportionally inter-related. Mechanical properties are functions of wood fibre properties, not those of the adhesive binder.
3. Particleboard and wood are linear elastic to at least 40% of short-term ultimate strength.
4. The modulus of elasticity of wood and particleboard is unchanged by mechanosorption for stress levels below 40% of short-term ultimate strength.
5. After nominally 40 days, all types of reconstituted wood panels under load in a sheltered exterior situation continue to deform at a common reducing rate.
6. In ambient conditions, deformation of wood and reconstituted wood panels appears to virtually asymptote in 12 to 15 months and then continues at a very much lower rate that depends on the size and frequency of moisture changes, until either failure occurs or a stable limit state is reached at which creep and relaxation cease.
7. All moisture changes create mechanosorptive effects. The period to failure is reduced by more frequent moisture changes, larger moisture changes or higher stresses. The naturally occurring but infrequent large changes in humidity increase deformation more rapidly than the more frequent smaller changes. In sum, the latter appear to increase deformation almost as much as the former.
8. Mechanosorptive deformation under load is directly proportional to the magnitude of a change in moisture content, but is independent of the rate of moisture change.

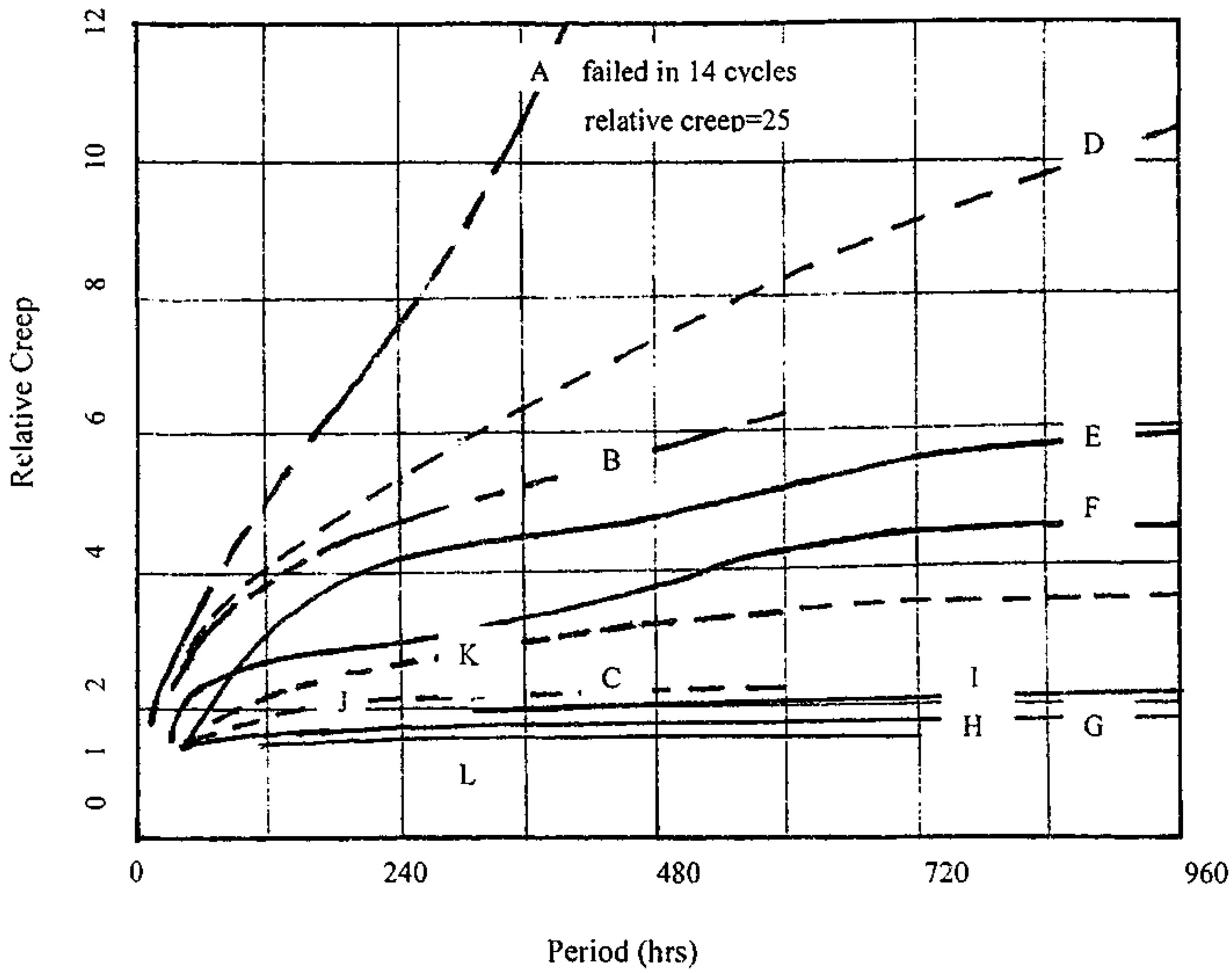
Mechanosorptive action is absent when the moisture gradient is zero irrespective of moisture content.

9. Mechanosorptive deformation is directly proportional to the level of externally applied stress at or below design stress levels.
10. Mechanosorptive deformation increases during the first adsorption period and all desorption periods, and recovers during all subsequent adsorption periods. Subsequent adsorption recovery is less than desorption increases.
11. The rate of viscoelastic creep in timber and particleboard is proportional to applied stress at or below design stress levels and, under similar moisture regimes, whether steady or fluctuating, is similar to that of plywood and nominally twice that of wood.
12. Viscoelastic and mechanosorptive creep differ, in that, on removal of the load, the former recovers with time, the latter with changes in moisture content.
13. It appears that the rate of increase in deformation mirrors the rate of stress relaxation, provided the applied stress approximates design stress level.
14. The relative creep of wood and particleboard in a sheltered external situation is nominally halved in an indoor situation.
15. The loss of strength that occurs in particleboard in an indoor situation is nominally 25% greater in a sheltered external situation.
16. In a sheltered external situation, the relative creep and loss of strength that occurs in particleboard appears to be nominally twice that of wood.
17. Most creep is recoverable when moisture is cycled.

18. A timber beam loaded at design stress levels in a sheltered situation, will exhibit, after removal of the load, no detectable residual deformation at the end of a period that is roughly six times the loading period.
19. Under steady conditions, wood loses less strength than the Madison curve predicts.
20. Before it fails, a loaded structural grade timber beam in an external situation will undergo a shortening along its compression edge that is 3 to 4 times the extension that occurs along its tension edge.
21. Under sustained load, timber with many defects has a lower relative loss of strength than timber with few defects.
22. Fatigue strength and mechanosorptive action are probably inter-related.
23. Ambient humidity induces in 19*mm* thick timber, infrequent moisture content changes no greater than 7% in a sheltered outdoor situation and 3% within a building, which correspond in 19*mm* particleboard to changes less than 6% and 2%.
24. The deformation of particleboard in ambient internal conditions equals that caused by fluctuating moisture content changes of 2%.

2.4.9 Graphical Representation of Creep and Relaxation

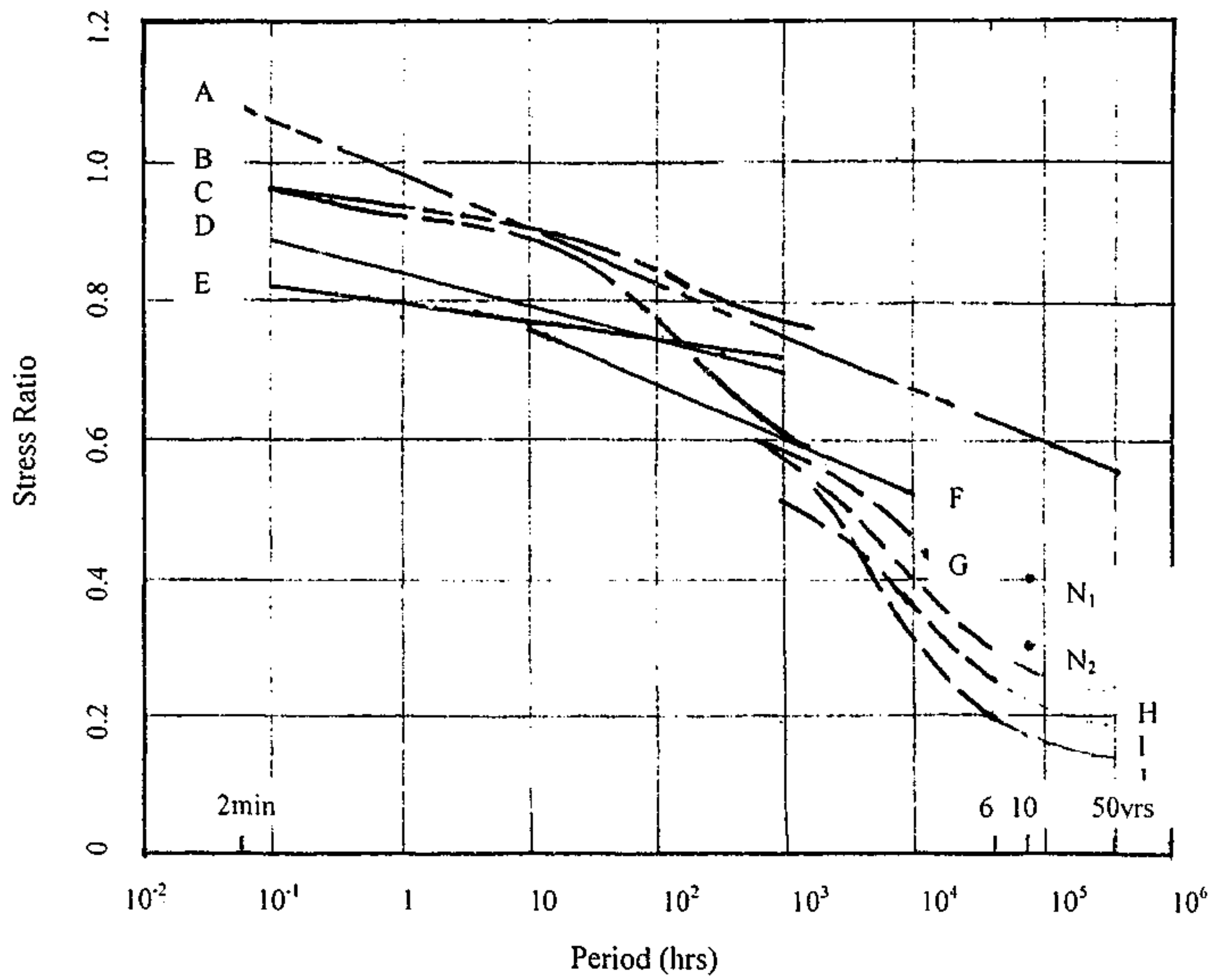
Several of the data on creep and relaxation in wood, particleboard and hardboard, referred to in the foregoing, are plotted in Figures 2.1 and 2.2 respectively to illustrate the range of test environments and stress levels investigated. Estimates of long-term creep are based on such data collected over periods in the order of 3 months, occasionally 2 years, except for a 5 year study by Dinwoodie et al (1992). Similarly, predictions of stress relaxation are based on short term data except for the 6 year observations by Hunt (1976).



Legend

A Clear Wood	Hearmon and Paton (1964)	stress ratio 0.375, cyclic 6-20%*mc*, 25°*C*
B Clear Wood	Hearmon and Paton (1964)	stress ratio 0.125, cyclic 6-20%*mc*, 25°*C*
C Clear Wood	Hearmon and Paton (1964)	stress ratio 0.375, 20%*mc*, 25°*C*
D Particleboard	Armstrong and Grossman (1972)	stress ratio 0.15, cyclic 6-18%*mc*, 25°*C*
E Hardboard	Armstrong and Grossman (1972)	stress ratio 0.15, 18%*mc*, 25°C
F Particleboard	Armstrong and Grossman (1972)	stress ratio 0.15, 18%*mc*, 25°C
G Particleboard	Armstrong and Grossman (1972)	stress ratio cyclic 0.1-0.6, 6%*mc*, 25°*C*
H Particleboard	Yang and Haygreen (1971)	stress ratio cyclic 0.1-0.6, 6%*mc*, 27°*C*
I Particleboard	Yang and Haygreen (1971)	stress ratio 0.1-0.6, 6%*mc*, 27°*C*
J Particleboard	Haygreen et al (1975)	stress ratio 0.1, 6-10%mc,27°*C*
K Particleboard	Bryan and Schniewind (1965)	stress ratio 0.3, 6-10%*mc*, 25°*C*
L Particleboard	Bryan (1960)	stress ratio 0.3, 6%*mc*, 22°*C*

Figure 2.1 Creep in Clear Wood and Particleboard



Legend

A	Clear Wood	Wood (1951) (Madison Curve)		
B	Clear Wood	Grosssman and Kingston (1963) Stress Ratio 0.37, 7% *mc*, 50°*C*		
C	Clear Wood	Grosssman and Kingston (1963) Stress Ratio 0.28, 12% *mc*, 21°*C*		
D	Particleboard	Bryan (1960) (As reinterpreted by Hunt 1976)		
E	Particleboard	Bryan (1960) Steady mc, 21°*C*		
F	Particleboard	M^C Natt (1975) Steady mc, 21°*C*		
G	Particleboard	Hunt (1976)	Indoor	680kg/m^3
H	Particleboard	Hunt (1976)	Sheltered Outdoor	720kg/m^3
I	Particleboard	Hunt (1976)	Sheltered Outdoor	680kg/m^3
J	Particleboard	Hunt (1976)	Sheltered Outdoor	600kg/m^3
N_1	Particleboard	Brynildsen et al (1976)	Indoor	660kg/m^3
N_2	Particleboard	Brynildsen et al (1976)	Sheltered Outdoor	660kg/m^3

Figure 2.2 Stress Relaxation in Clear Wood and Particleboard

2.4.10 Rheology Modelling

In the endeavour to describe creep and stress relaxation from first principles, various models of rheological behaviour have been developed. Several of these are discussed below.

2.4.10.1 The Madison Curve

The Madison curve has its basis in work by Kitazawa (1947) who proposed that $F = F_t(1 - m\log t)$ where the ratio of applied force to force $\frac{F}{F_t} = k_1$ after period under load, t, and m is a wood characteristic. Commenting on this work, Grossman (1954) believed that *"for the general description of stress relaxation in wood, a linear relationship between stress and (log) time is inadequate, and a relaxation spectrum must be used."*

Wood (1951) observed for two years the behaviour of defect free 25mm square specimens of Douglas fir subject to bending over a span of 355mm under ambient conditions. A linear regression of stress relaxation against log time was fitted to the observations resulting in the duration-of-load expression

$$k_1 = 0.904 - 0.063\log_{10} t \qquad 2.1$$

where k_1 is the ratio applied stress/short-term ultimate strength.

t period under sustained load in hours.

This expression is known as the Madison curve. Wood noted that at 55%rh the predicted relaxation was almost identical to that at 90%rh, consequently the influence of ambient moisture variations that occurred during the test period is reflected in the expression. The effect of this influence is imprecise in that it is related to time rather than the size and frequency of moisture variations. It is also imprecise because the expression is derived for defect free wood, whereas lower quality structural grade timber with lower short-term strength appears to lose less strength proportionally so that the duration of load effect has relatively less influence.

2.4.10.2 Internal Bond Energy Models

In the internal bond energy approach, deformation is taken as the sum of elastic deformation due to the stretching of bonds and crystalline cellulose chains, and viscous flow due to bonds within the wood cell wall being displaced from one energy well to an adjacent well. Creep deformation is regarded as accumulated shear slippage between individual elements, and that stress relaxation at constant strain is equal to the shear slippage between individual elements that reduces overall stress in the wood. In recognizing that creep and relaxation are qualitatively similar, Robson (1988) believed that they can be described, using molecular kinetic rate theory, by equations that take a similar form and proposed that the rate

for creep is given by

$$\frac{d\varepsilon}{dt} = \frac{V_h}{V_m k e^{\frac{V_h f_h}{2kT}}} \qquad 2.2a$$

and for relaxation by

$$\frac{d\varepsilon}{dt} = \frac{V_h}{V_m k e^{\frac{V_h f_h}{2kT}}} \qquad 2.2b$$

where

- σ is the stress on the viscoelastic system
- ε creep strain
- T °Kelvin
- V_m volume of moving cell wall element
- V_h volume swept by moving cell wall element

f_h driving stress

k Boltzmann's constant

Robson (1987) concluded that while these expressions can describe steady state creep they fail to account for creep recovery during moisture adsorption and accelerated creep during changes in temperature. That is, mechanosorptive effects due to ambient humidity variations are not embraced.

2.4.10.3 Viscoelastic and Mechanosorptive Models

Over decades, various arrangements of springs and dashpots have been developed to model viscoelastic creep, and more recently nechanosorptive creep. Springs represent the elastic crystalline cellulose and dashpots the non-elastic non-crystalline matrix. In the matrix, viscoelastic and viscous behaviour considered to be time dependent, and mechanosorptive behaviour moisture dependent.

Leicester (1971) and Ranta-Maunus (1973) proposed an expression for creep that took variations in moisture into account as

$$\frac{d\varepsilon}{dt} = \frac{1}{E}\frac{d\sigma}{dt} + (\alpha + m\sigma)\frac{du}{dt} \qquad 2.3$$

α coefficient for free moisture expansion

m material coefficient for mechanosorptive creep

du moisture change

Subsequently a number of researchers predicted that a form of strain hardening exists which limits viscoelastic and mechanosorptive effects on long-term deformation so that a stable limit state is reached. Hunt and Shelton (1988) proposed that a mathematical series which represented strain and included mechanosorptive creep resulting from moisture change would take the form

$$\varepsilon = \sum_{n=1}^{n=\infty} f(du, n, \sigma) \tag{2.4a}$$

and give

$$\varepsilon_L = J_\infty \sigma \left[1 - e^{-a\int_0^t du(t)} \right] \tag{2.4b}$$

from which Hunt (1991) expressed the strain limit as

$$\varepsilon_L = \alpha J_\infty = \sigma \left[J_E + J_0 + J_1\left(1 - e^{-n/N_1}\right) + J_2\left(1 - e^{-n/N_2}\right) \right] \tag{2.4c}$$

where

ε_L is strain at creep limit under given stress

σ stress

a characteristic wood coefficient

J_∞ creep compliance at limit state

J_E elastic deformation

J_0 creep compliance that occurs before moisture cycling

J_i creep compliance for a characteristic moisture cycle

N_i characteristic moisture cycle

n number of moisture cycles

Hunt (1991) assumed that such a stable limit state, related particularly to cellulose molecules in the cell wall S2 layer, which forms some 85% of the wood cell wall, is reached when stress is below 10% of short-term ultimate strength. Within dry interior conditions, defined as 30%*rh*, and subject to low stress, defined as less than $0.75 \times 10^{-3} E$ which is nominally 10*MPa* for timber, Hunt (1991) believed that viscoelastic strain is linear with time. But under high stresses that cause irreversible damage, creep is non-linear and therefore a stable limit state is unlikely to be reached before failure occurs. Equations 2.4 reproduce with considerable accuracy the graphical representation of the changes in strain developed in wood subjected to moisture cycling under a constant stress of 3/8 of short-term ultimate strength due to Hearmon and Paton (1964) as shown in Figure 2.1.

One model, Yahiaoui (1991), consists of a number of Kelvin bodies in series and assumes that creep at constant stress and varying moisture content tends to a stable-state limit, and at the stable-state limit, the type of stress affects the moisture-shrinkage coefficient. Compared with the unloaded condition, creep is considered to be smaller in tension and greater under compression.

Toratti (1992) developed equations for the stable strain limit as a function of stress, moisture change and time which gave the stable limits,

ε_T, under tensile stress as

$$\varepsilon_T = \frac{0.7\sigma}{E}\left(1 - e^{-2.5\int_0^t du(t)}\right) \qquad 2.5a$$

and ε_C, under compressive stress as

$$\varepsilon_C = \frac{0.7\sigma}{E}\left(1 - e^{-2.5\int_0^t du(t)}\right) + \frac{0.1\sigma}{E}\int_0^t du(t) \qquad 2.5b$$

The omission of the second term from equation 2.5a assumes that the mechanosorptive effect differs between tension and compression. The omission of viscoelastic effects from both equations, assumes that they have no influence on the limit state. Neither omission fits the literature. From the available experimental evidence, Toratti

(1992) concluded, as did Hunt (1991), that *"the ratio between the increment of mechanosorptive creep and the amount of moisture change tends to decrease as moisture accumulation increases,* [but] *it is hard to say if mechanosorptive creep approaches a limit value....."*

Lu and Leicester (1997) proposed that $\varepsilon := \frac{\sigma}{E}\left(1 + k_{creep}\right)$ 2.6a

where k_{creep} is a function of moisture and climate induced moisture change.

Mundy et al (1998) referred to the expression for creep in particleboard developed by Dinwoodie et al (1991) in which no explicit provision for mechanosorptive effects is included. The deflection limit, Y_t, at time t, was given as

$$Y_t = \beta_1 + \beta_2\left(1 - e^{-\beta_3 t}\right) + \beta_4 t^{\beta_5} \text{ when } 0 \le \beta_5 \le 1 \qquad 2.7a$$

which gave

$$\varepsilon_t = \frac{\sigma}{E_1} + \frac{\sigma}{E_2}\left(1 - e^{-\frac{E_2 t}{\eta_2}}\right) + \frac{\sigma t^{\chi}}{\eta_3} \qquad 2.7b$$

where

σ is applied stress

ε_t strain at time t

β_i deformation compliance

E_1 modulus of first spring

E_2 modulus of viscoelastic spring

η_2 viscosity of viscoelastic dashpot

η_3 viscosity of final dashpot

χ viscous modification factor

Mundy et al (1998) concluded that the equation is a good fit to observed behaviour for the first six months but is only a reasonable fit to 12 years of observations, and that "*to produce an acceptable balance between the contributions of elastic, viscoelastic and viscous behaviour is unrealistic.*"

Hanhijarvi and Hunt (1998) found that as mechanosorptive creep accumulates, the viscoelastic creep rate decreases, and that recovery in mechanosorptive creep decreases the rate of recovery in viscoelastic creep. This suggested to them that the two mechanisms are independent processes.

2.4.10.4 Damage Accumulation Models

A typical damage accumulation model, that includes the effect of cyclic moisture change, was developed by Fridley et al (1992) who believed that stress relaxation mirrored creep.

$$\frac{d\alpha}{dt} = e^{\left(-A+B\sigma+C\left[\frac{dm}{dt}\right]t_m\right)} \tag{2.8}$$

where α is the damage accumulated. (α =1 at failure).

m: ratio moisture content change to original moisture content.

σ relaxation at the reference moisture content.

t_m period of moisture change

A, B, C, model parameters

The parameters were given constant values of; A=27, B=24.1, C=92.9, the large effect of cyclic moisture change is reflected in the high value of C. The model is deficient in that damage will accumulate in the absence of an applied stress.

2.4.10.5 Moisture Content and Moisture Change

It is apparent that both moisture content and moisture changes affect viscoelastic creep. and Moisture change affects mechanosorptive deformation and the distribution of moisture throughout sections of timber and quantification of rates of diffusion is essential if long term deformation is to be accurately assessed.

As part of an investigation into mechanosorptive effects, Lu and Leicester (1997) examined moisture diffusion in structural sized timber. The deformation of timber under stress induced by mechanosorption is the sum of elastic and mechanosorptive deformation

$$\varepsilon = \varepsilon_E + \varepsilon_{ms} \qquad 2.9a$$

with the incremental mechanosorptive creep, $\Delta\varepsilon_{ms}$ given by

$$\Delta\varepsilon_{ms} = \frac{\sigma}{E}\Delta\alpha \qquad 2.9b$$

where $\Delta\alpha$ is the incremental moisture change

Assuming moisture in wood fibre changes cyclically and sinusoidally,

$$m = m_{av} + m_0 \sin pt \qquad 2.9c$$

then

$$\Delta\alpha = 2km_0 \times n \qquad 2.9d$$

where m_0 is half the amplitude of the moisture change

m_{av} average moisture content for the period t

n number of moisture changes during the period t

Lu and Leicester (1997) proceeded to develop an equation that should "*imitate the true induced temperature and humidities in.....range of climate zones in Australia*" and afford a reasonable approximation of $\Delta\alpha$.

2.4.10.6 Particleboard Relaxation

Hunt (1976) fitted cubic regression curves to observations of stress relaxation for three particleboards in a sheltered outdoor situation (SE2) as shown earlier on Figure 2.2. Typical of the equations was that for board with a density of $680 kg/m^3$.

$$\log t = 168.908 - 327.8369(\log R) + 219.407(\log R)^2 - 49.373(\log R)^3$$

where L is period to failure in hours.

R strength at t hours / short term ultimate strength in %.

This expression predicted a relaxation in strength to 17.3% after 50 years.

2.4.11 Discussion

It is evident that all species of wood and all reconstituted wood panels are rheologically similar, but exhibit quantitative differences in behaviour under load due to the nature of their respective component bonding. The range of stress likely to be met in timber structures in service is in the order of 15% to 25% of short-term ultimate strength under dead load, rising to around 35% under dead and live load. Within these limits, the effect of ambient conditions on particleboard, hardboard and plywood is similar to its effect on wood, that is, higher moisture contents and wider or more frequent moisture fluctuations enhance both creep and stress relaxation.

The literature puts forward three explanations for creep deformation and stress relaxation in wood and reconstituted wood panels. One is that both are due to viscous flow caused by breaking and reforming molecular bonds within the cell wall under the action of stress, which action is exacerbated by the migration of water molecules during changes in moisture content. Another is that they are due to anatomical restructuring of the wood cell, also under the action of stress and also exacerbated by changes in moisture content. A third combines both anatomical and molecular bonding effects with moisture change.

In ambient conditions and in any sheltered situation, the large creep of all reconstituted wood panels during the initial 600 hours or so under load is irrecoverable, and unrelated to either initial elastic or subsequent viscoelastic and mechanosorptive deformations. The disproportionately high initial creep rates are related to bonding mechanisms that are unique to the particular adhesive. Some experiments show the initial creep of phenolic bonded boards to be thrice that of urea bonded boards and about 50% more than that of melamine bonded boards. When moisture content variations reach a balance with ambient variations in humidity, the differing initially high creep rates reduce to a single rate that is common to all boards, irrespective of the adhesive type. Whereas initial creep rates are particular to the adhesive type, the subsequent stabilized rate is governed by wood fibre characteristics alone. The stabilized rate is directly proportional to moisture content and double the rate in wood. High initial creep rates are virtually removed by moisture induced stress relief that breaks weak adhesive bonds prior to installation.

The strength and moduli of elasticity of wood with a moisture content around 15% are lowered some 20% to 25% by saturation. But particleboard exhibits such rapidly increasing mechanical deterioration once its moisture content exceeds 12% for prolonged periods that structural use in unsheltered situations could not be contemplated without a reliable paint coating that virtually prevents the accumulation of moisture within the wood. Standard specifications in current use for particleboard rely on a tensile test applied normal to a soaked specimen's surface to indicate retention of bond integrity, and hence mechanical properties, in the presence of moisture. None require a mandatory surface treatment that impedes moisture penetration.

Studies of the effect of heat alone on the creep of wood and reconstituted wood panels are limited to ambient conditions. Much of the work has been carried out over ranges of temperature and moisture content that make it impossible to separate the interwoven effects of heat and moisture change. The evidence largely supports the view that, for practical purposes, the effect of heat alone on the mechanical properties of wood and particleboard is negligible in normal ambient conditions. The changes in moisture content that accompany temperature variations must create mechanosorptive effects, but they are not taken into account explicitly. It appears that the lower creep and relaxation rates induced by reductions in moisture content that accompany a rise in temperature, largely offset the higher rates due to the added heat energy. Salmen (1991) demonstrated that, within ambient conditions, paper desorption is linearly related to rising temperature and adsorption to falling temperature.

However, the work on particleboard is confusing, suggesting that strength reduces significantly and linearly with rising temperature in the ambient range on the one hand, and on the other, that ambient temperature variations have no effect. In part, the conflicting data may be attributable to experimental methods whereby specimens are held in conditions where constant humidity is maintained but temperature is uncontrolled, rather than the intended conditions where constant moisture content is maintained. General relationships between relative humidity, moisture content and temperature for particleboard and wood are plotted in Figure 3.1, reference to which indicates that a rise of 12°*C* reduces the moisture content of both wood and particleboard by around 1%.

Particleboard subjected to fatigue loading for ten million cycles, retains around 45% of its short-term ultimate tensile and shear strengths. Tension and shear failure rates for physically similar boards follow precisely identical curves and are independent of the adhesive type or board density. This suggests that shear failures are induced by principal tensile stresses acting across wood cell lamella, adding weight to the creep hypothesis put by Christensen (1962) and others as discussed in para. 2.4.7.1. Boyd (1982) holds that failures are primarily anatomical due to principal compressive stresses that act parallel to the cell lamella and induce forces sufficient to cause cell walls to buckle and consolidate. Whilst a similar effect could be expected in random particleboard, where restraint to buckling is offered to particles by others that are oriented laterally and adhered to them, it is unlikely to be significant and more likely that compressive strength is increased to match tensile strength.

Hunt (1976) noted that particleboard in sheltered exterior sites, attains maximum moisture contents of about 7%, which are halved indoors. The retained strength of particleboard in sheltered exterior sites rises indoors by around one stress grade, 25%, when humidity remains nominally constant at 50%*rh* and by about two stress grades, 50%, when held below 50%*rh*.

Bryan (1960), Dinwoodie et al (1991) concluded that the deformation rate in bending of particleboard is about double that of plywood, which ratio probably held irrespective of moisture content cycling in the normal ambient range.

Creep in particleboard and timber that occurs in protected exterior environments is nominally halved indoors. The annual deformation rate of structural timber after 13 years under load is around 7% when fully exposed to weather and 4% indoors.

Preston (1974) confirmed that crystalline cellulose chains are impermeable to moisture. It appears therefore, that they are unaffected by moisture content or moisture movement. However, in the non-crystalline cellulose, hemicellulose and lignin matrix, it appears that moisture movement weakens the cell structure. These phenomena might explain the retention of a constant modulus of elasticity whilst strength, particularly compressive strength, continues to reduce under mechanosorptive action.

2.4.12 Conclusions

Particleboard is a reliable structural material when used in sheltered outdoor situations where its moisture content averages no more than 12%, and does not exceed 18% except for occasional periods of limited duration. The structural reliability particleboard in any sheltered situation is equal to that of structural timber.

The effect of heat on particleboard is similar to its effect on timber in ambient conditions. With average moisture contents below 15% for wood, or 12% for particleboard, ambient temperature variations have a negligible effect on mechanical properties.

The effect of moisture on timber and particleboard indicates a need for defined service conditions and their regionalization in terms of climatic conditions. Proposals for these are put forward in Chapter Three.

The mechanisms that govern mechanical behaviour are common to wood, plywood, hardboard and particleboard, but a fundamental rheological explanation of them remains elusive. All current mathematical models of creep and stress relaxation with the passage of time are wanting in that they are unable to reliably predict behaviour beyond periods of continuous observation. The literature suggests they are interdependent. It also suggests on one hand that both are time related and on the other that they depend primarily on the level of moisture and the size and number of moisture changes experienced during service. And it also appears that, at stress levels normally met in service, a limit state may be reached at which, for practical design purposes, both cease. An investigation of stress relaxation in particleboard and defect free wood forms the subject of Chapter Four.

2.5 STRUCTURAL RELIABILITY

The design of timber structures in Australia shifted from working stress methods to limit states design methods in 1997 with the replacement of AS 1720.1-1988, *Use of timber in* *structures*, by AS1720.1-1997. Uncertainty in the capability of limit states methods to reliably predict true structural behaviour is reflected in values ascribed to the capacity factor ϕ referred to in equation 1.2. No capacity factors specific to the use of structural particleboard are nominated in AS 1720.1-1997, hence, the evaluation of appropriate values of ϕ for particleboard elements in structures forms an integral part of this study.

An appreciation of the reliability theory supporting limit states design methods is a necessary precursor to assessing reliability factors for particleboard. The literature concerned with the reliability theory supporting AS1720.1-1997 is reviewed in detail in Chapter Five. It reveals an anomaly between material characterization and limit states design methods that arose from an ambiguity between values of the capacity factor employed in each. It is shown, that the anomaly is removed by removing the capacity factor from material characterization.

CHAPTER THREE

SERVICE ENVIRONMENTS AND CLIMATE

3.1 INTRODUCTION

Climatic conditions raise two issues with respect to the mechanical behaviour of wood and reconstituted wood products. One is the reduction in strength that occurs when rising humidity induces higher moisture contents, particularly when they exceed the seasoned condition. The other is the mechanosorptive action that reduces strength and increases deformation with changes in moisture content induced by ambient variations in humidity and temperature. The effects of humidity are of particular relevance to particleboard, the greater sensitivity of which to high moisture contents compared with timber, necessitates that ambient climatic conditions encountered during service are suitably classified and regionally delineated. This enables the effects of moisture changes induced by ambient conditions to be applied to structures with respect to their geographical location.

Unlike European structural design codes that classify service environments for wood and reconstituted wood products in terms of humidity, AS 1720.1-1997 makes, no detailed provisions for the effects of humidity. It refers only to humidity in the context of the general effects of moisture contents ranging between unseasoned (green) and seasoned (15-16%*mc*) for timber structures in all regions north of latitude 16°S and coastal regions north of latitude 25°S. AS 1720.1-1997, therefore, is of limited use as is the European classification, which fails to embrace the dryer climate more widely met in Australia.

In this chapter, an extended classification that includes dryer conditions is introduced and regional boundaries of the service environments are mapped for Australia. A moisture content factor is derived as a substitute for the temperature factor specified by AS 1720.1-1997.

3.2 SERVICE ENVIRONMENTS

3.2.1 Definitions

To suit perceived climatic conditions, European design codes divide the environments, within which a timber structure may serve, into three classes. They are defined in terms of the prevailing humidity at 20°*C* as follows:

Service Class SC1 would not exceed 65%*rh* except for a few weeks in any one year.
Service Class SC2 would not exceed 85%*rh* except for a few weeks in any one year.
Service Class SC3 could exceed 85%*rh* for any period.

These are described as typically representing respectively, a building interior, a sheltered outdoor position, and a fully exposed situation. No reference to ranges of humidity or temperature within a service class is made.

At 20°*C*, most species of timber attain equilibrium moisture contents (emc's) of 12% and 18% in SC1 and SC2 conditions respectively, and particleboard attains corresponding values of 8% and 14%. As revealed by the literature, exposure of particleboard to relative humidity much in excess of 85% for prolonged periods leads to excessive creep. Hence, structural application in the European Class SC3 environment is unlikely to be acceptable unless the board is protected by an effective, properly maintained paint treatment, a qualification too uncertain to allow general use. AS/NZS 1491-1996, *Finger jointed structural timber*, adopts the three European classes and definitions, however, in regions with drier climates where humidity is generally much lower, the writer believes that the single class, SC1, is too limiting to properly facilitate structural use, not only of particleboard, but also of timber.

3.2.2 Australian Conditions

Mapping by the Australian Bureau of Meteorology indicates that relative humidity in Arnhem Land, Cape York and coastal regions north of Townsville, fluctuates normally between 60% and 90%, and sustains 70% - 80% for prolonged periods. In limited areas of the highlands of New South Wales, Victoria and Tasmania it ranges between 40% and 80%, occasionally reaching 90%. The lowest humidity is confined to the arid central region of Australia where it rarely falls below 20%*rh* and seasonally varies on average 30%, fluctuating between 15%*rh* and 50%*rh*,.

Regions in which maximum variations in relative humidity, on average 40%, occur, are the highlands and along the southern mainland coast where, respectively, it ranges seasonally from 35% - 40% to 75% - 80%.

Except for the northern regions described above, periods of high humidity in Australia are less extreme and of shorter duration than those prevailing in Europe. For example, Hunt (1976) recorded external relative humidity in London ranging between 44% and 90%, a variation of 46%. Thelandersson and Sandahl (1994) reported a similar range in Lund, also in a coastal region on a latitude some 4° north of London.

Consequently, four service environments are viewed as more generally suited to the use of timber and reconstituted wood products. These are designated herein as, SE0, SE1, SE2, and SE3, in which at 20°*C*, the average maxima of the ruling ambient relative humidity are respectively 45%, 65%, 80% and 90% with related exceptional maxima attained occasionally of 65%, 80%, 90% and 90%+. By comparison with the European classifications, it will be noted that there is no equivalent to SE0, that the environment defined by SC1 and SE1 is the standard regime specified for testing the mechanical properties of all wood and wood-based products, and that SC2 approximates SE2, but gives higher emc's than SE2. SC3 and SE3 are common.

3.2.3 Regional Delineation

To enable adjustments to be made for reinforcement corrosion, the concrete structures code, AS 3600-1994, *Concrete structures,* divides Australia into three Climatic Zones with the aim of regionally delineating the climatic affects that debilitate reinforced concrete. These are the Temperate zone, which covers most of the populated country, the less populated Tropical zone, and the relatively unpopulated Arid remainder.

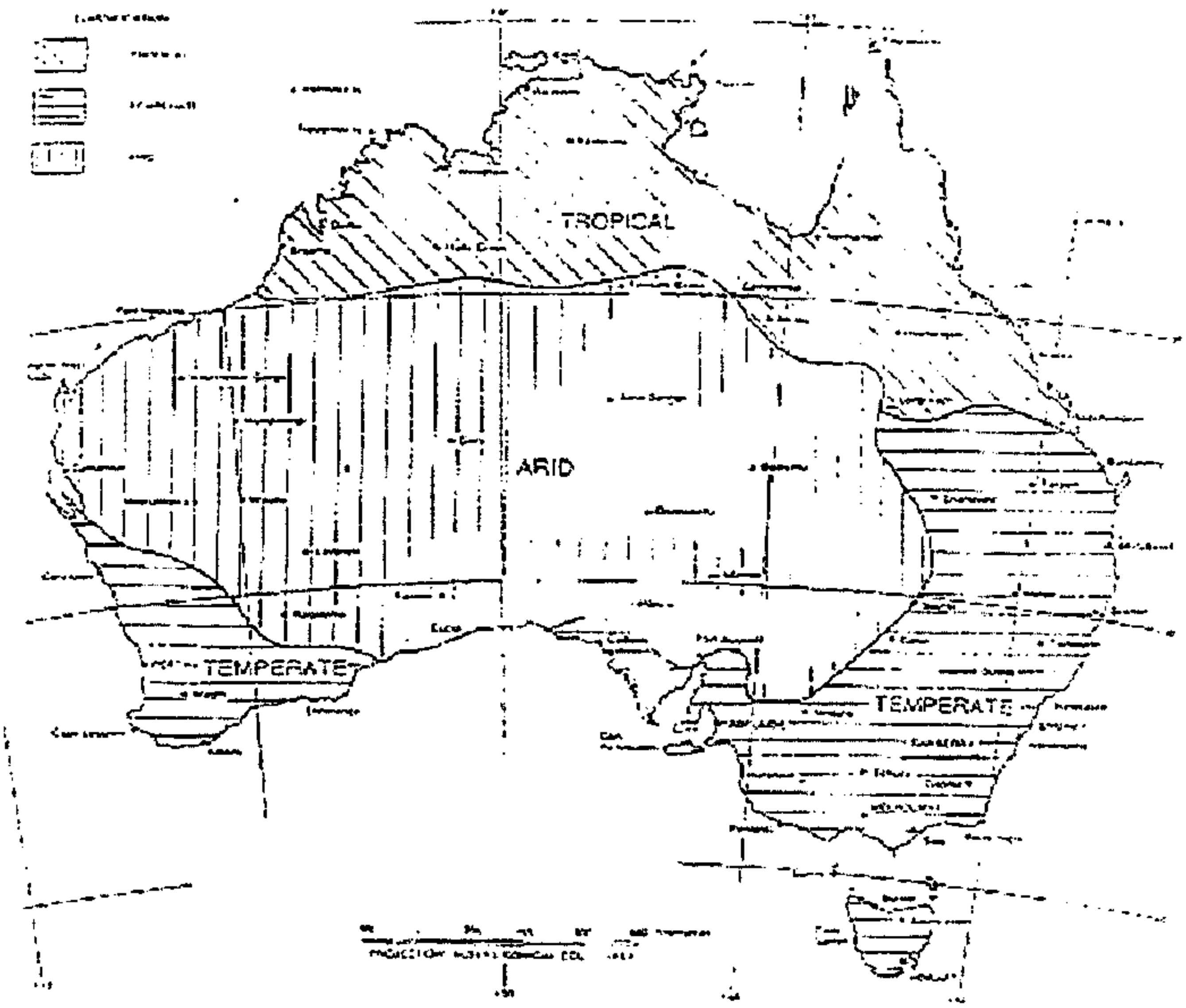


Figure 3.1 Climatic Zones in Australia for Concrete Structures.

(from AS 3600-1994)

The zoning delineated in AS 3600-1994, although familiar to engineers, is not entirely suited to timber and wood composites and is in need of further refinement to account for seasonal variation in humidity and the sustained high humidity in regions north of the Tropic of Capricorn. Whilst the data for timber referred to in the foregoing discussion are not in entire agreement, it is sufficiently consistent to delineate approximate boundaries for the regions in which the four service environments, SE0–SE3, occur. Approximate boundaries for these regions are shown in Figure 3.2 and should be treated with due caution for structural design purposes. Established local humidity records, when available, must over-ride the mapped boundaries.

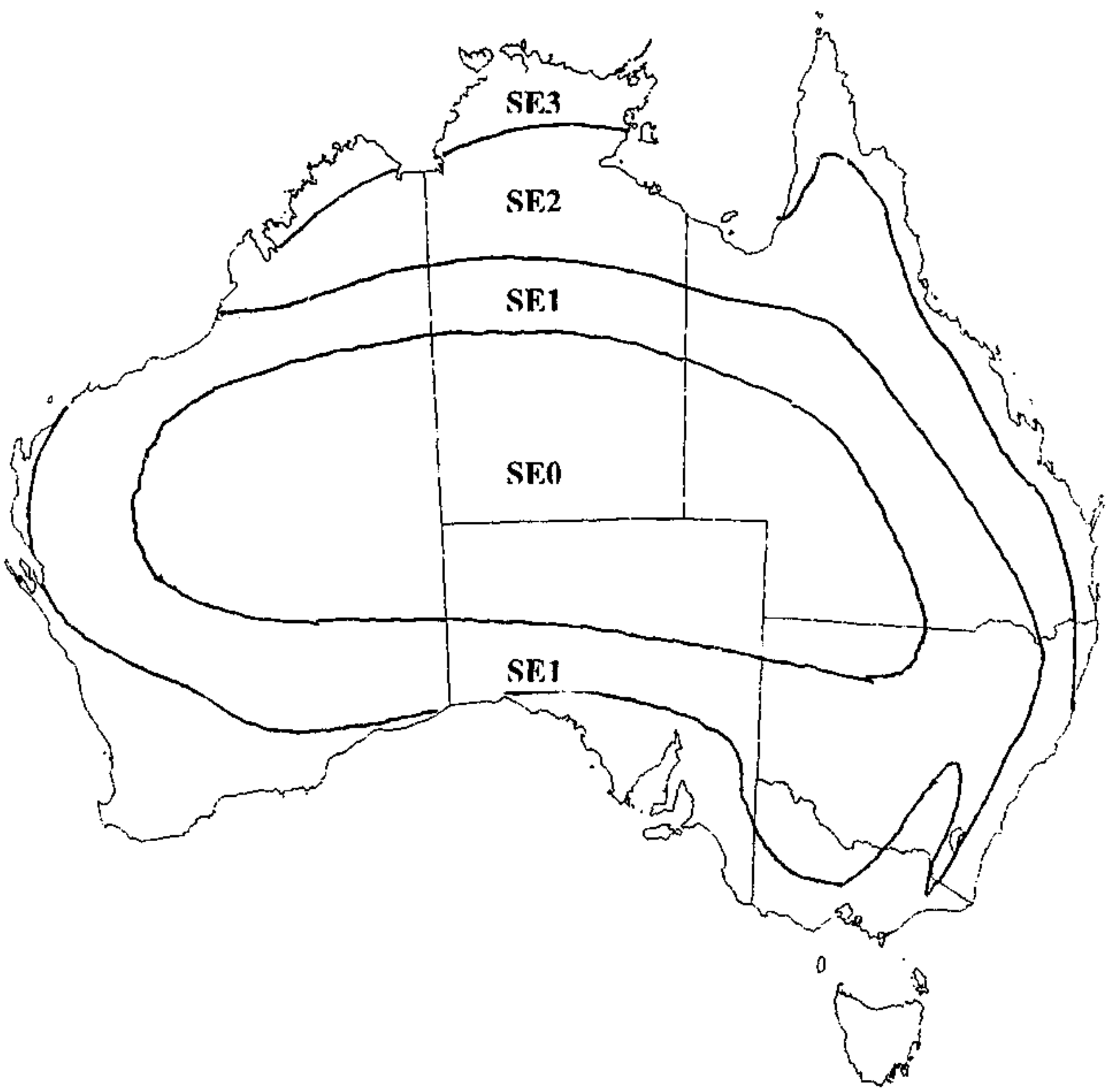


Figure 3.2 Regional Zones of Service Environments, SE0-SE3, in Australia for Timber Structures.

It will be noted that most building structures are located in either the SE1 region or the SE2 region below latitude 25°S, the significance of which is discussed in Chapter Four.

3.3 MOISTURE CONTENT AND HUMIDITY

3.3.1 Equilibrium Moisture Content v Relative Humidity

The Wood Handbook (1974) tabulates emc's of wood against relative humidity and temperature stating that for most practical purposes it applies to any species. Halligan and Schniewind (1972) plotted a similar relationship for particleboard at 72°*F* (22°*C*) to which has been added further curves gathered from isolated data due other researchers.

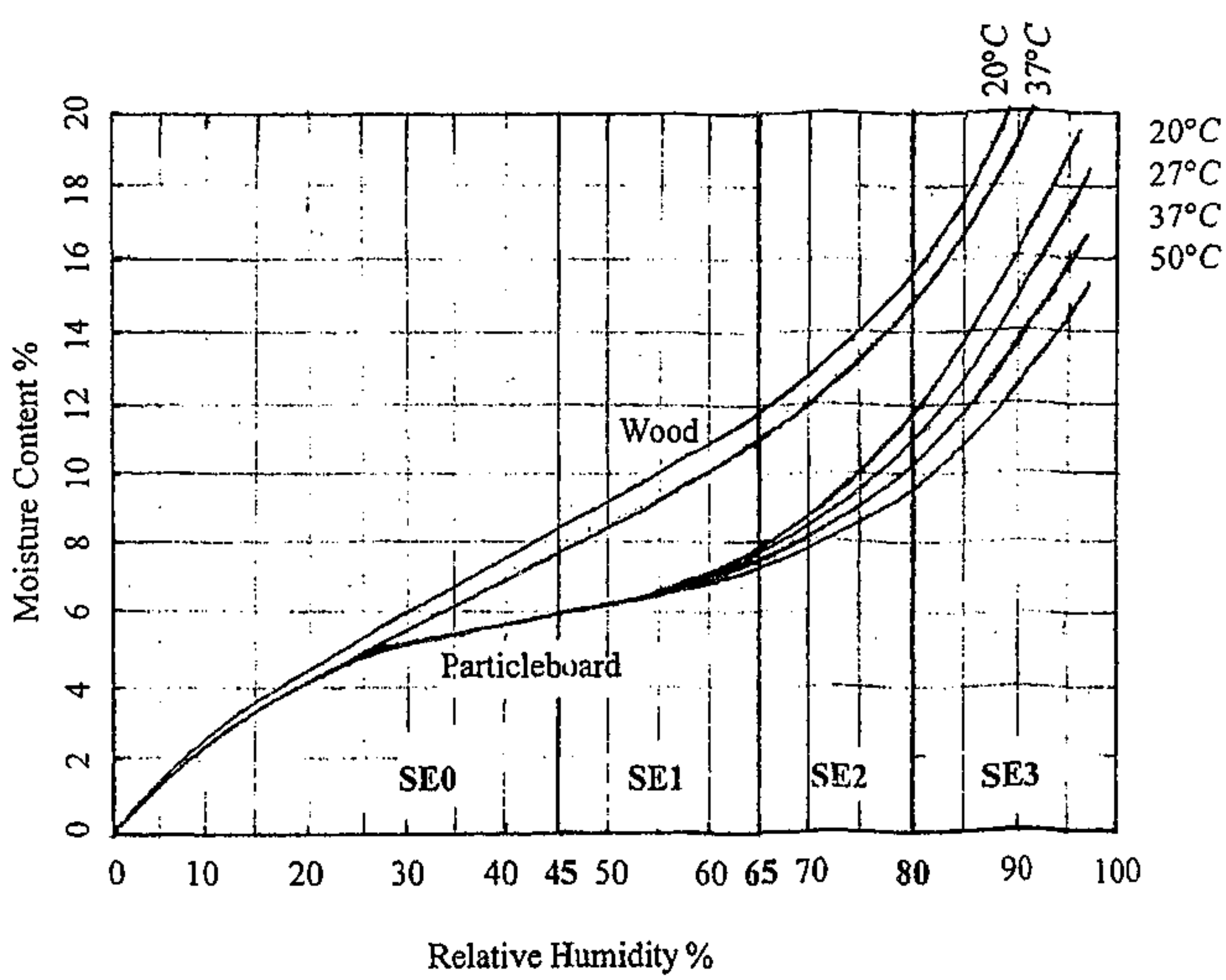


Figure 3.3 Equilibrium Moisture Contents v Relative Humidity for Wood and Particleboard (after Wood Handbook 1974, Halligan and Schniewind 1972 and oths.)

3.3.2 Sheltered External Environment.

Over a three year period, Finighan (1966) measured the variation in moisture content of eight species of seasoned wood placed under sheltered external conditions in several locations across Australia. Specimens were *6mm*, *19mm*, and *45mm* thick, and of the eight species, the annual variation in moisture content was greatest for P. radiata. A similar study by Bragg (1986) did not include P. radiata, but, from a comparison with other species common to both his and Finighan's studies, Bragg obtained slightly lower emc's than Finighan. The boundaries for Queensland, Figure 3.2, are based on the more detailed information collected by Bragg (1986).

Finighan found that the maximum moisture content attained annually by the P. radiata specimens was highest in the northern mainland coastal regions as typified by Innisfail and Cape York where it reached 20%. Along the southern mainland coastal region, the highest moisture contents were found in Adelaide, Melbourne and Perth, where all three recorded 7%*mc*

Hence, to make a conservative comparison with the moisture contents that would be attained in *19mm* particleboard made from P. radiata, the data for *19mm* thick specimens of solid P. radiata extracted from Finighan (1966), was used for this study. Using Figure 3.3, the writer reduced the solid wood data to the equivalent, but lower, emc's for particleboard. In Innisfail and Cape York and the sub-tropical coastal regions generally, particleboard would attain a maximum moisture content around 15% for short periods, and a moisture content approaching 9% for lengthy periods. Along the southern mainland coastal region, a maximum moisture content of 6% would be attained annually, accompanied by a maximum variation of 4%.

In Central Australia, arid conditions produce, in a sheltered external situation, the lowest humidity averaging between 20%*rh* and 45%*rh*, occasionally rising to 55%. These induce emc's in wood ranging normally between 4.5% and 8.5% and occasionally 10% giving a maximum annual variation of 5.5%. In particleboard, by reference to Figure 3.3, corresponding emc's lie between 4% and 6%, rising occasionally to 6.5%, giving a maximum annual variation of 2%.

Armstrong (1983) and Lhuede (1992) concluded that, in a sheltered situation and a locality, classified as an SE1 environment where timber normally has an average moisture content around 10%, it develops a maximum average annual variation approaching 4%, accompanied by daily variations of 1.5%. In this environment, (SE1), particleboard acquires an average maximum moisture content close to 7% and similar annual and daily variations.

In summary, in any sheltered external situation over most of Australia, moisture contents developed in particleboard do not exceed a ruling average maximum of 12% and occasionally 16%. Hence, the emc's attained by particleboard in sheltered conditions in Australia do not militate against satisfactory structural performance.

3.3.3 Internal Environment.

Taken from CSIRO (1968) data, the average and range of moisture contents for wood situated indoors in Australia are given in Table 3.1. The tabulation shows that, whether in a heated or fully air-conditioned building, average moisture content of wood is in the order of 2% less than that in an uncontrolled building, but the range remains somewhat the same for both.

Table 3.1 Moisture Content of Wood for Indoor Conditions. (CSIRO 1968)

Location	Air conditioned Average / Range	Winter Heat Average / Range	Uncontrolled Average / Range
Adelaide	9 / 7-10	8 / 7-8	9 / 7-12
Brisbane	10 / 8-12	11 / 8-12	11 / 10-12
Canberra	8 / 5-10	8 / 5-10	11 / 9-12
Hobart	-	7 / 6-9	11 / 10-12
Melbourne	9 / 7-10	9 / 7-10	11 / 10-13
Perth	9 / 8-10	9 / 8-10	10 / 9-12
Sydney	9 / 7-11	10 / 7-12	11 / 10-12

It is evident that conditions within controlled buildings in Australia and Europe tend to be similar as they reflect levels of humidity and temperature suited to human comfort, respectively, 45-60%*rh* and 20-22°*C* rather than external ambient conditions. In a heated room in London, Hunt (1976) recorded a range of relative humidity identical to that experienced in Australia, which is 40%*rh* to 56%*rh*. The corresponding moisture contents in particleboard are 5.5% and 6.5%, the variation 1%.

Using Figure 3.3, the tabulated values for wood indicate that moisture contents in particleboard would average 7% in an uncontrolled building and 6% in a controlled building, and range, on average, 2% in both. These moisture contents are nominally equal to those attained in the sheltered external environment, (SE1), but the variation within a building is halved.

3.3.4 Service Environment v Equilibrium Moisture Content

By reference to Figure 3.3, at 20°*C* in the service environments, SE0-SE3, wood attains respectively average ruling emc maxima of 8%, 12%, 16% and 21%, rising occasionally to 12%, 16%, 21% and more than 21%. Particleboard attains respectively, ruling maximum emc's of 6%, 8%, 12% and 16%, rising occasionally to 8%, 12%, 16% and more than 18%. The emc's of solid wood and particleboard related to external relative humidity at 20°*C* and 37°*C* in service environments, SE0-SE3, are given in Table 3.2.

Table 3.2 Service Environment v Moisture Contents of Timber and Particleboard at 20°*C* and 37°*C*.

Service Environment	Ambient %*rh* Ruling / Occasional	Moisture Content %, 20°*C* Ruling / Occasional / Change		Moisture Content %, 37°*C* Ruling / Occasional / Change	
		P'board	Timber	P'board	Timber
SE0	45 / 65	6 / 8 / 2	8 / 12 / 3	6 / 8 / 2	9 / 11 / 3
SE1	65 / 80	8 / 12 / 4	12 / 16 / 4	8 / 11 / 3	11 / 15 / 4
SE2	80 / 90	12 / 16 / 4	16 / 21 / 5	10 / 14 / 4	15 / 19 / 5
SE3	90 / >90	16 />18 / >5	21 />21/ >5	14 />14 / >4	19 />19 / >5

Note: Ambient %*rh* refers to the maximum relative humidity that generally rules and to the exceptions that occur occasionally.

It is apparent that wood is appreciably less responsive to humidity below 65%*rh*, (SE0, SE1) than above 65%*rh*, (SE2, SE3) and particleboard's response is less again. The gain in strength when moisture contents fall below 15% for timber, or 12% for particleboard, is small and of little practical significance. However, of primary significance is the large reduction in strength and increase in deformation caused by mechanosorption during changes in moisture content, as distinct from that due to high moisture contents alone.

The maximum annual change in the emc of particleboard is 4% and occurs in SE1 and SE2 environments, the minimum of 2% occurs in an SE0 environment. Daily variations are unlikely to exceed 1%. The determination of appropriate factors to account for mechanosorptive effects of these changes in moisture content forms the subject of Chapter Four.

3.4 TEMPERATURE AND MOISTURE CONTENT.

Whether in wood or particleboard, Figure 3.3 reveals that, at any humidity level below 80%*rh*, which embraces the normal ambient range, an increase in temperature of 10°*C* reduces moisture content by around 1% and vice versa. As an example, whilst no moderation of external conditions takes place within uninsulated, light timber framed and timber-clad buildings, the damping exerted by a brick veneer cladding moderates internal temperature to a limited extent. For instance, at *6pm* during February in Melbourne, typical maximum internal temperatures in these buildings were respectively 36°*C* and 31°*C*, which, by reference to Fig. 3.3, could reduce moisture contents by around 0.5%.

AS 1720.1-1997 applies a "temperature" factor k_6=0.9 to the characteristic strength of wood and plywood in covered timber structures that are located in coastal regions of Queensland north of latitude 25°S and in all other non-coastal regions north of latitude 16°S. These regions fall into the SE3 service classification, in which relative humidity reaches 80% for prolonged periods and raises the emc of wood above 16%. Because in ambient conditions the mechanical properties of wood and particleboard are unaffected by temperature alone, Hearmon and Paton (1964), Dinwoodie et al (1991), it is evident that the factor implicitly reflects the reduction in strength due to the higher moisture contents that accompany the lengthy periods of high humidity experienced in these regions. Thus, the factor k_6=0.9 accounts for the loss in strength as moisture content approaches saturation, at which level, strength is around 25% less than that with emc's below 15 or 16%.

With respect to particleboard, investigations by Hunt (1976) show that in the SE2 environment, where the ruling maximum emc is 12%, occasionally reaching 16%, a factor of k_6=0.9 would apply to board with a density of 720kg/m^3, decreasing to 0.8 for 680kg/m^3 board. A value of 0.7 is extrapolated for 650kg/m^3 board. Values of the temperature / Moisture Content Factor, k_6, that appear to be appropriate for particleboard are given in Table 3.3

Table 3.3 Temperature/Moisture Content Factor, k_6, for Particleboard

Structural Grade*	Density kg/m^3	Temperature/Moisture Content Factor k_6
PB16	650	0.7
PB20	680	0.8
PB25	720	0.9

*Refer to Chapter Six for classification of Structural Grades.

It is noted that, the lower emc's attained at higher temperatures compared with those at 20°C indicate that as a lower limit, particleboard retains its structural capacity in an extreme regime of 37°C and occasional 95%rh (16%mc). In this rare environment, one that would not be encountered for prolonged periods in properly constructed and ventilated timber framed buildings, the retained strength adds an inherent margin of safety against poor building practice that may contribute to excessive moisture take-up occasionally.

3.5 MOISTURE GRADIENTS

Moisture gradients throughout a section of wood or particleboard induced by climatic changes create mechanosorptive effects. They are difficult to quantify because rates of moisture diffusion are time related and vary according to wood species, material thickness and density, and vapour pressure. The issue adds another layer of complexity to the second objective of this study, viz., the long-term response of particleboard to climatic conditions.

In an endeavour to account for the effect of timber thickness on moisture gradients and hence on mechanosorption, Lu and Leicester (1997) devised an expression that would reasonably "*imitate the true induced temperature and humidities in ... a range of climatic zones in Australia*". With respect to temperature variations, as discussed in paragraph 3.4, normal ambient temperature effects are inseparably interwoven into humidity variations and when considering the effects of thickness on climatically induced mechanosorption, the comparatively minor effects of temperature are encompassed within the major effects of humidity.

However, when considering the effects of thickness on ambient moisture content variations, Finighan (1996) found that the moisture content of P. radiata specimens 19*mm* thick was practically the same as that of 6*mm* thick specimens and rarely 1% more than that of 45*mm* thick specimens. Because the majority of timber framing is not more than 45mm thick, it would appear that the effects of moisture gradients in timber framing generally could be conservatively represented by specimens 19mm thick. Similarly, by reference to Figure 3.3, it is concluded that the climatic response of particleboard, would be practically the same for all thicknesses between 12*mm*, and 22*mm*.

Consequently, by confining the investigation into the long-term behaviour of particleboard and timber to material that is 19*mm* thick only, makes it possible, for the purposes of this study, to ignore the effects of diffusion and observe the effects of mechanosorption in the limit.

CHAPTER FOUR

LONG TERM MECHANICAL BEHAVIOUR

4.1 INTRODUCTION

Of major concern to the engineer designing timber structures is that predictions of the long-term strength of members based on the widely used Madison curve, equation 2.1, are probably unsafe. Fortunately however, deformation criteria generally govern the size of members, and it seems probable that predictions of long-term deformation are reasonably reliable. This is evidenced by the observation that catastrophic failures occur rarely and when they do, are due to overloading low grade material, or to poor design and construction. The primary aim of this aspect of the study is to clarify the issue as it affects particleboard by establishing a value for stress relaxation under sustained load for 50 years that can be compared with that currently inferred for structural grade sawn timber from the Madison curve for clear wood. And in so doing, possibly gain some better insight into stress relaxation in timber.

It was concluded from the literature, that the rheological behaviour of wood and reconstituted wood products under stress is yet to be explained. The principal difficulty is whether deformation and stress relaxation reach a stable limit. This investigation illustrates fairly conclusively, that

- both defect free timber and particleboard, stressed respectively to 60% and 30% of short-term ultimate strength in ambient conditions, appear to reach a state where stress relaxation and deformation practically cease,
- mechanosorptive action is not time dependent, but entirely due to moisture changes induced by ambient conditions,
- mechanosorption is the dominant cause of stress relaxation.

As a consequence, the predicted duration-of-load factor for stress relaxation in structural grade sawn timber is considerably lower than that given by the Madison curve.

A further outcome of the experimental approach is the development of a comparatively rapid method for determining maximum stress relaxation in defect free wood and particleboard. It is probable that the method has general application to other reconstituted wood composites with low variability and possibly to sawn timber with different types of defects.

4.2 POSTULATES

4.2.1 Duration of Load Effects

Estimates given in the literature for the residual strength of particleboard after 50 years under load in a sheltered external (SE3) environment vary significantly from 0.2 to 0.4 of short-term ultimate strength. Whilst the lower estimates of strength stem from research of much longer duration than that supporting the higher values, the large difference between them is probably fundamental in origin and appears to rest with the mechanosorptive phenomenon.

There is strong evidence that, under the action of external forces that create internal stresses of sufficient magnitude, wood material is progressively weakened and deformed by mechanosorptive effects. Depending on the size and number of ambient humidity cycles experienced, and on the level of stress to which the wood material is subject during the cycles, cumulative irrecoverable reductions in strength and increases in deformation occur. Apparently, cycles of humidity will occur occasionally that induce moisture content changes large enough to weaken the wood material irreparably if the stress is sustained at a sufficiently high level. Accompanying the occasional large changes are more frequent smaller changes that add to the cumulative weakening effect. Acting simultaneously with mechanosorption is a viscoelastic action that is time dependent and also reduces strength and increases deformation under sustained stress.

It is commonly held, although not demonstrated, that the rate of deformation progresses rapidly to asymptote for all practical purposes in 12 or 15 months. If so, mechanosorption is the dominant action thereafter, but it is unknown whether mechanosorptive and viscoelastic actions reach a limit state after which no further relaxation or creep will occur provided the level of sustained stress is sufficiently low. The limit would become apparent when, under the action of cyclic moisture changes, absorptive deformation is equal and opposite to desorptive deformation.

4.2.2 Isotropic Effect

The random orientation of wood chips imparts in-plane isotropy to particleboard. In bending normal to the surface, it is therefore probable that the adhesion of chips to one another increases their resistance to compressive stress by inhibiting cell wall buckling. If so, it seems that failures are as likely to originate in the extreme tensile fibres as in the extreme compressive fibres.

4.2.3 Postulates

From the foregoing three postulates emerge:

1. Wood and particleboard subject to sustained stress in ambient conditions in sheltered situations, lose strength, principally, by mechanosorptive action on the wood fibre. The loss can be conservatively quantified by considering, alone, the cumulative mechanosorptive actions.
2. Wood and particleboard subject to sustained stress around the level normally experienced by structures in service, in ambient conditions, in sheltered situations, reach a stable limit state at which stress relaxation and creep cease
3. The process of chipping and randomly reorienting the wood fibre in particleboard increases its resistance to in-plane compressive stress so that, in-plane, compression strength approximates tension strength.

4.3 OUTLINE OF INVESTIGATION

4.3.1 Aim

The aim is to verify the postulates and develop a simple empirical expression that will predict with reasonable accuracy the reduction of strength that occurs in 19*mm* thick wood and particleboard in sheltered situations when subject to sustained stress around normal service level for 50 years. It is not intended that the work should provide a rheological explanation of mechanical behaviour.

At loads around normal service level, that is, around 0.3 short-term ultimate strength, experimental problems arise in accurately measuring, within the small total strains developed under low stresses, the very much smaller increments of strain induced by changes in humidity. Typically, these involve difficulties associated with specimen matching, differing short-term ultimate strengths, dimensional variations and statistical manipulation. To reduce these difficulties the approach adopted in this study is to;

- Determine the elapsed period to failure of particleboard under high but constant levels of bending and tensile stress in ambient conditions in a sheltered external situation and indoors,
- Transform the maximum ambient humidity variations observed into a typical wave form that can be reproduced cyclically in an environmental chamber,
- Determine the elapsed period to failure of particleboard and defect free timber when held under low levels of tensile stress and subject to forced humidity cycles that replicate the typical wave form,
- Determine whether a limit state is reached and if so, at which stress level, and
- Fit to the results an equation that predicts stress relaxation.

4.3.2 Investigation Stages

The investigation was made in two stages.

The first stage, commencing in 1998 and completed in May 2002, consisted of loaded specimens subject to natural climatic conditions while held both indoors and under shelter outdoors. The aims were to establish

1. sufficient data that would enable stress relaxation in particleboard held indoors and in a sheltered outdoor situation to be extrapolated to provide a value after 50 years under sustained stress
2. the effect of in-plane isotropy on the compressive strength of particleboard.

With respect to the first aim, no valid extrapolations for stress relaxation is possible without consideration of the cumulative effects of moisture sorption and rates of moisture diffusion through the material under test. With respect to the second, results confirm that the third postulate is correct and, therefore, it is unnecessary to perform both bending and tensile tests to appreciate mechanosorptive effects. Consequently, the second stage of the investigation was confined to specimens subject to tensile loading only.

The second stage of the investigation stemmed from phenomena observed in the first stage which appeared to confirm that the period to failure depends not on the passage of time, but solely on the size and number of humidity changes experienced and the level of stress to which the material is subject. This observation tied neatly into experimental work published at that time, (Milner et al, 1999), that indicated the possible presence of a limit state at which stress relaxation ceases. The second stage commenced in 1999 and completed in August 2002, consisted of loaded specimens subject to forced humidity changes that cyclically reproduced maximum natural humidity variations while held in a climate chamber. In the second stage the aims were to establish whether

1. large changes in humidity cause failure in particleboard subject to sustained high tensile stress.
2. it is possible to reach a limit state in particleboard beyond which mechanosorptive effects cease to cause stress relaxation when subject to sustained tensile stress that approximates dead load design stress level.
3. it is possible to reach a similar limit state in defect free timber subject to tensile stress around twice dead load design stress level.

The second stage experiments demonstrated that large changes in humidity cause rapid weakening and, fairly convincingly, the probability that it is possible to reach a stable limit state.

4.3.3 Experimental Stress Levels

4.3.3.1 Duration-of-Load Factors

To limit the extent of the work for the second stage of the investigation, it was necessary to ascertain appropriate experimental levels of stress. Reference was made to the literature concerning stress relaxation and creep because mechanosorptive effects appear closely related to each other for both particleboard and wood. The matching form of stress relaxation and creep curves for wood is referred to by Grossman and Kingston (1954, 1963), Grossman (1971), and others and is implicit in the theoretical models currently being considered, Toratti (1991), Hunt (1991), and others. Haygreen et al (1975) and Hall et al (1977) inter-related creep and relaxation behaviour in particleboard and confirmed its qualitatively similar behaviour to wood.

The effects of load sustained for periods up to 5 or 6 years can be drawn from the literature for stress relaxation in particleboard, Hunt (1976) and wood Toratti (1992), and up to 13 years for deformation in timber, Kingston (1968), but beyond these periods they are inconclusive. Fortunately, around Melbourne, a small number of timber structures constructed with primary elements of hardboard and particleboard can be examined, some of which seen over 30 years service. From the foregoing it should be possible to glean some insight into suitable stress levels for long-term experimental work. These are explored in the following discussion.

4.3.3.2 Duration-of-Load Factors Drawn From The Literature

Hunt (1976) estimated that, after 50 years under permanent load, particleboard with a mean density of $720 kg/m^3$ in a protected exterior situation, (SE2), retains around 0.19 of its short-term ultimate strength. The data indicate that retained strength increases in an internal environment, (SE1), by possibly 25% to around 0.25 of short-term ultimate strength. The stress relaxation value of 0.19 refers to board manufactured c1969 with the same average density and situated in the same environment but with average short-term strength some 25% lower than the less variable board currently manufactured. It is probable therefore, that long-term loss of strength, being lowered by reduced variability, is possibly reduced by 25%, in which event k_1 is raised to 0.24. It is also possible that the predictive curve developed by Hunt (1976) gives a conservative estimate.

At the other end of the scale, the Nordic Committee on Building Regulations (1973) expected particleboard with a density not less than $500 kg/m^3$ under sustained load to retain 0.40 of short-term ultimate strength in European Service Class SC1 and 0.25 in Class SC2, (SE2). From McNatt (1975) it is concluded that, in SC1 conditions, particleboard retains around 0.40 after 50 years. Pearson (1977) estimated that particleboard with similar density and mechanical properties to that tested by Hunt (1976) would have a residual strength of 0.41 when held under a steady, but higher humidity that gave a moisture content around 11% as attained in steady relative humidity of 75%*rh*. Haygreen et al (1975) concluded that the creep of particleboard due to ambient moisture changes within a building is similar to the creep developed under a steady moisture content of 8% as attained at 65%*rh*, (SE2). The indoor conditions prevailing during investigations by Hunt (1976) closely approximated SE2 conditions, and Hunt's relaxation estimate is thus complemented and reinforced by Haygreen et al (1976). In 1985 Leicester, (in Hanley et al 1985), proposed a factor of 0.37 for particleboard that corresponds reasonably to the Nordic 'indoor' (SC1) value of 0.40. But these are some 50%, or two stress grades, higher than the indoor value of 0.24 predicted by Hunt (1976). The increase is difficult to explain, except in terms of the reduced variability and complementary 25% increase in mechanical properties of particleboard since 1969 when Hunt's test boards were manufactured.

The demonstrated rheological similarity of particleboard and hardboard provides further useful data. Work on hardboard by Kloot 1954 resulted in working stresses of around 0.24 of short term ultimate strength being proposed for a temperate sheltered environment, but they remain unpublished. This value is in substantial agreement with work by Ramaker and Davister (1972) and Brynildsen et al (1976). More recently, duration-of-load factors for hardboard and particleboard were specified in BSS 5268, Part 2-1996, to suit working stress design methods. When modified to suit limit states design methods for a 50year sustained loading, the factor for hardboard is 0.28 and for particleboard, 0.43. This conflicts with Perkitny and Perkitny (1966) who regard stress relaxation in particleboard as around 80% of that in hardboard, which reduces k_1 for particleboard to practically the same value. Whilst the hardboard factors in BSS 5268, Part 2-1996 agree substantially with the literature, the duration-of-load factors for particleboard appear somewhat high. Duration-of-load factors modified for limit states design use are given in Table 4.1.

Table 4.1 Duration-of-Load Factor, k_1, for Strength Limit States Design use adapted from Working Stress Design Values specified by BSS 5268.2-1996.

Loading	Period	Hardboard k_1	Particleboard k_1
Dead + permanent imposed.	Long Term	0.28	0.43
Dead + temporary imposed.	Medium Term	0.67	0.74
Dead + imposed + wind.	Short Term (Std. Test)	1.00	1.00
Dead + wind gust.	Very Short Term	1.33	1.26

4.3.3.3 Duration-of-Load Factors Drawn From As-built Structures.

In Australia, investigation into the structural use of particleboard was preceded a decade earlier by similar work on hardboard .The writer, c1960, undertook, in an unheated warehouse situated in Melbourne, in which neither humidity or temperature was recorded, some full-scale short-term prototype load testing on hardboard webbed box beams. The outcome indicated that a duration-of-load factor of 0.24, (Kloot 1960), would enable economical structures incorporating primary hardboard elements to be built in a sheltered environment. Various structures with hardboard webs were subsequently constructed, a number of which were inspected recently and found to be performing satisfactorily. Leicester, c1964, used the same factor when designing an 8*m* span floor and roof supported by hardboard webbed members for large extensions to the CSIRO Division of Forest Products (DFP) laboratories in South Melbourne, (CSIRO, Forest Products News, 1964). The laboratories saw some 15 years of service before demolition when the DFP timber mechanics group moved to the Division of Building Research (DBR) at Highett. Leicester, c1970, designed large rafters with webs of particleboard for CSIRO's air-conditioned Highett laboratories. These too show no distress after more than 30 years of service. From this historical evidence, and the demonstrated rheological similarity of hardboard and particleboard, it is not unreasonable to infer for particleboard under sustained load for 50 years in a sheltered external (SE2) environment, a stress relaxation factor of 0.30.

4.3.3.4 Possible Limit State in Wood

With respect to the possible existence of a stable limit state, Milner et al (1999) showed that creep strain in defect free wood under a compressive stress of 10.4*MPa* asymptoted after some 200 cycles of humidity that varied between 25%*rh* and 90%*rh* over 15 minute periods, while oscillating consistently with amplitudes around 150μ. The evidence suggests that the specimens reached saturation, the migration of water molecules and hence mechanosorptive action ceased, and that a limit state was probably reached. The stress of 10.4*MPa* is in the order of the design dead load stress that a saturated structural grade timber member would normally sustain in service where dead load forms about a third of the total action effect. Assuming strength is lognormally distributed, the stress is nominally doubled for clear wood. When the stress was raised to 18.4*MPa* the amplitudes nominally doubled, but the experiment was stopped before a limit state was reached. However the indications are that under a stress around 0.6 of clear wood ultimate strength, a limit state is possible.

To support this stress level, reference is made to the likely relaxation factor of 0.3 for particleboard discussed previously. Due to the strongly orthotropic nature of wood, strength perpendicular to the grain is comparatively negligible, commonly taken as 2.5% of that parallel to the grain and strength is thus highly directional. Laufenberg (1984) measured, in particleboard, the effective strength of the constituent randomly arranged wood chips, and demonstrated that the randomised orientation creates a virtual in-plane isotropic condition which reduces strength to almost half that of an equivalent section of timber irrespective of direction. Consequently, when particleboard is subject to the same average cross-sectional stress, the actual stress in the wood fibre that is offering effective resistance is nominally double that in defect free solid timber. Being proportional to stress, stress relaxation in particleboard made from wood essentially free of major defects, for example, the excessive sloping grain in branch-wood, is approximately double that of defect free wood. Hence, consistent with the factor of 0.3 for particleboard, it appears that the duration-of-load factor for defect free timber in a sheltered external environment may possibly rise to at least 0.6. And therefore, that the attainment of a stable limit state in defect free timber under a sustained stress of 0.6 or more of short-term strength appears possible.

4.3.3.5 Possible Stable State Stress Levels

From the foregoing, the stable state stress levels that appear possible for the second stage of the experimental work are 0.3 and 0.6 of the respective short-term strength of particleboard and defect free wood.

4.4 EXPERIMENTAL WORK

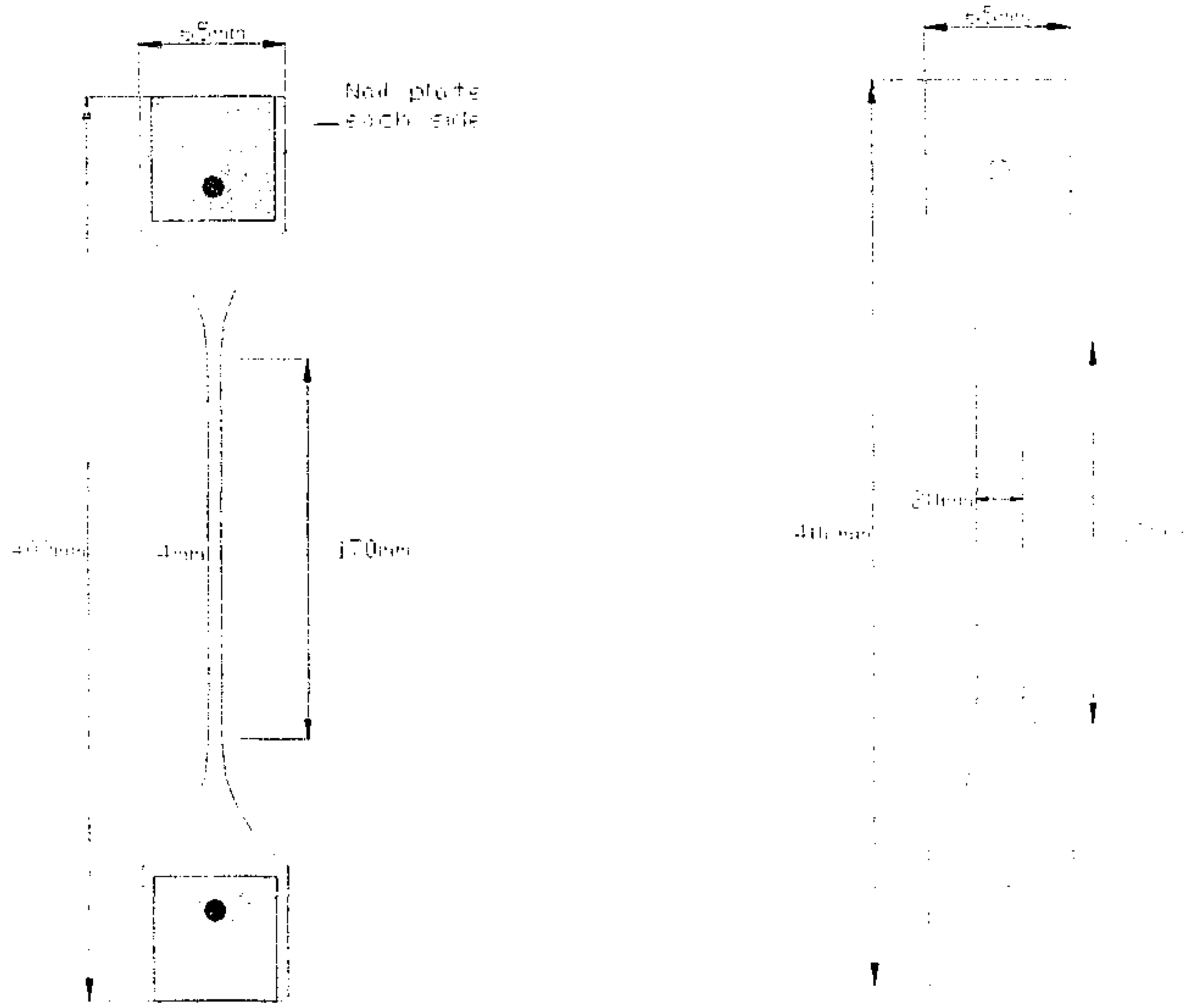
4.4.1 General Description

Specimens of 19*mm* thick flooring grade particleboard conforming to AS 1859-1998 were subject to Experiments 1 to 5. Experiment 1, took place indoors in natural climatic conditions. Experiment 2, which is a continuation of Experiment 1, was conducted beneath an outdoor shelter in natural climatic conditions. Experiment 3 took place in the small climate chamber used by Milner et al (1999) that accommodated single specimens, 4 and 5 in a climate chamber large enough to accommodate multiple banks of specimens. The climate chambers were programmed to deliver forced humidity variations that represented the maximum natural humidity condition measured in Experiment 1.

Specimens of 19*mm* thick defect free P. radiata were subject to Experiment 6. This took place simultaneously with Experiment 5 in the large climate chamber.

Simultaneously, the behaviour of similar specimens of particleboard and defect free P. radiata similarly loaded was monitored in a steady environment of 20°*C* and 65%*rh*.

4.4.2 Specimens



P. radiata **Particleboard**

Figure 4.1 Tension Specimens

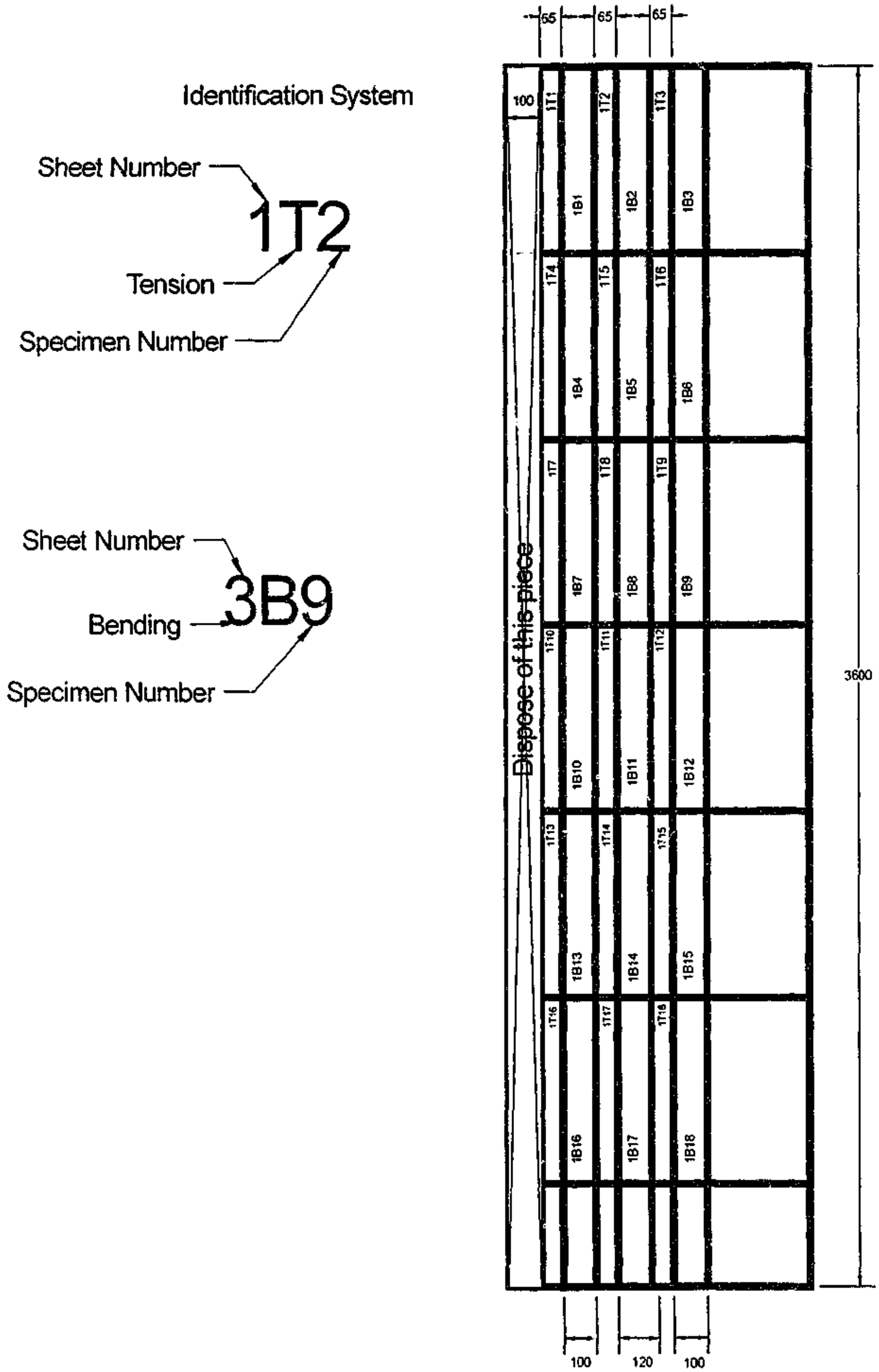


Figure 4.2 Cutting Pattern

4.4.3 Test Apparatus

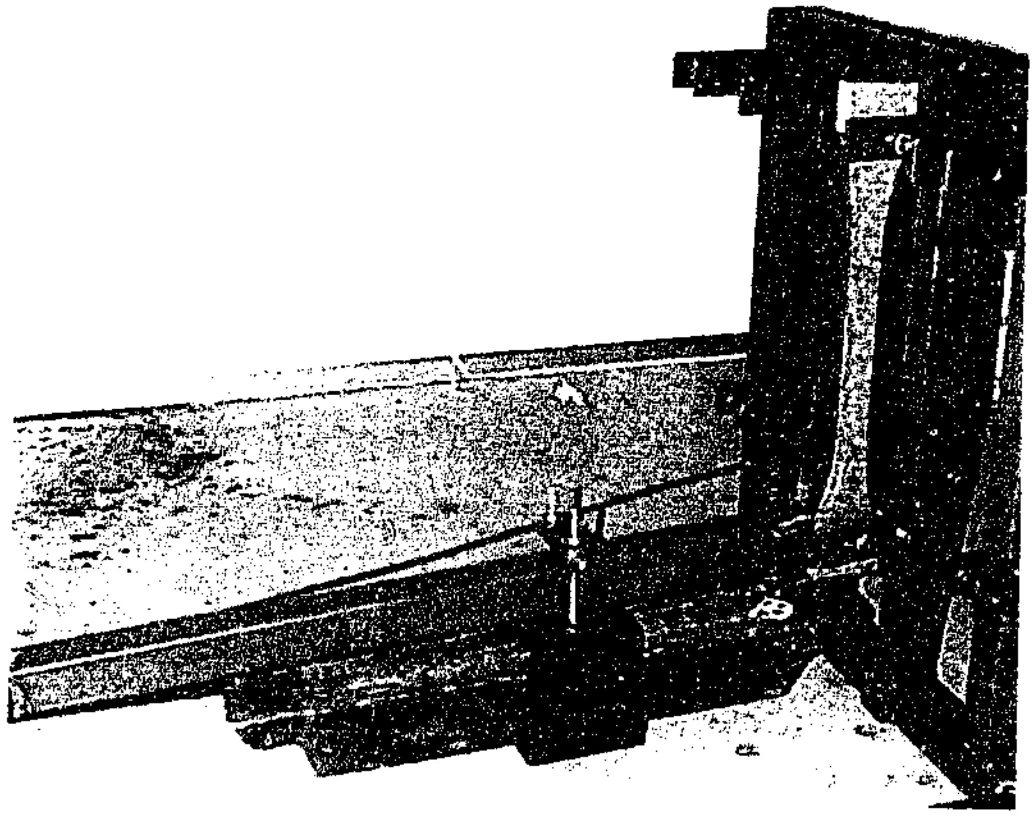


Figure 4.3 Tension Test Rig

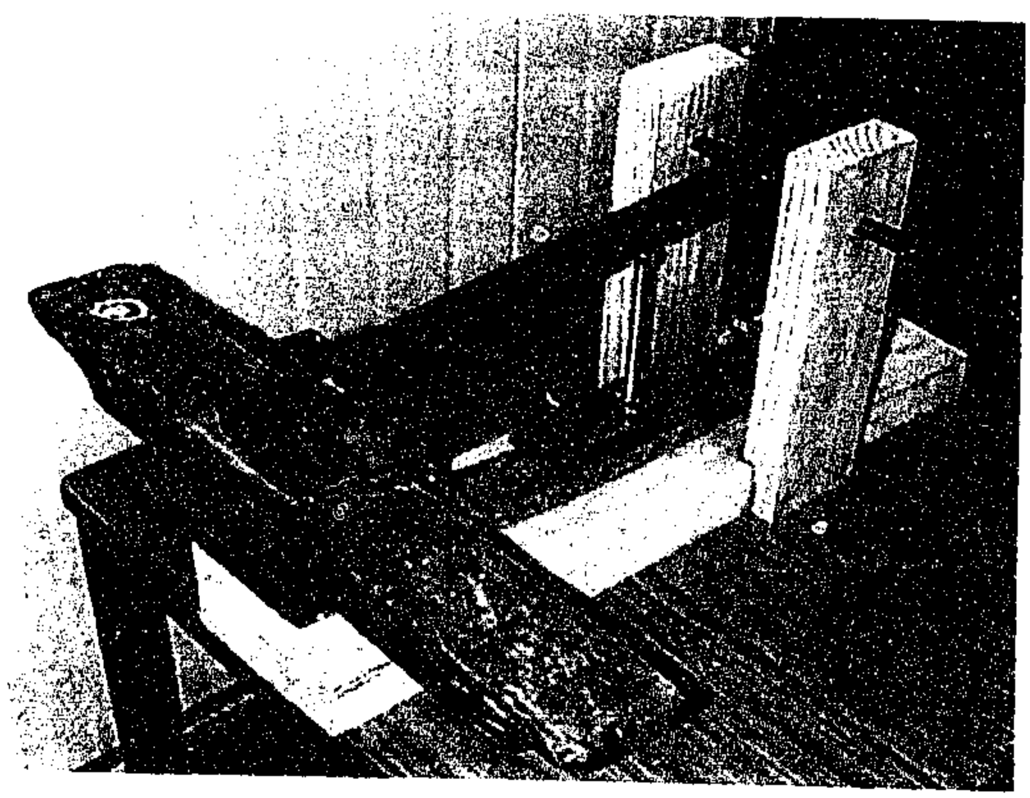


Figure 4.4 Bending Test Rig

4.4.4 Experiment 1

Aim

To determine the elapsed period to failure of 19*mm* particleboard, loaded in bending to 75% and 60% of short-term modulus of rupture, and in tension to 75% and 60% of short-term ultimate strength, when placed in natural ambient conditions in an internal situation.

To determine whether the random chip orientation in particleboard imparts in-plane isotropy that increases resistance to compressive stress so that failures in bending normal to the surface tend to originate in the tensile fibres.

Method

The work was conducted in a 3.5*m* square test room, the floor of which was a suspended concrete slab housed within a large steel framed, steel sheeted, unlined and unheated building. The walls of the test room were 19*mm* particleboard and the ceiling a 50*mm* fibreglass blanket. Ambient humidity and temperature in the test room were continuously recorded on a paper hygrograph.

Eighty tension and forty bending specimens were cut to the shapes shown in Figures 4.1 and coated with silicone over their sawn edges; their two (original board) surfaces remained uncoated. After conditioning at 0.65*rh* at 20°*C*, the short-term ultimate strength of each specimen to be placed under long-term loading was obtained by breaking its matched pair in the appropriate tension or bending rig intended for the long-term test, shown respectively in Figures 4.3 and 4.4. The remaining sixty specimens, conditioned similarly, were installed rapidly in loading rigs in the test room on 20 March 1999. Twenty specimens were loaded in tension to 0.75 of their individual short-term ultimate strength and twenty to 0.60. Ten specimens were loaded in bending to 0.75 of their individual short-term ultimate modulus of rupture and ten to 0.60. Elapsed periods to failure were logged electronically for the 0.75 loading and manually for the 0.60 loading.

Results

Elapsed periods to failure, inhours, are recorded on Figure 4.5.

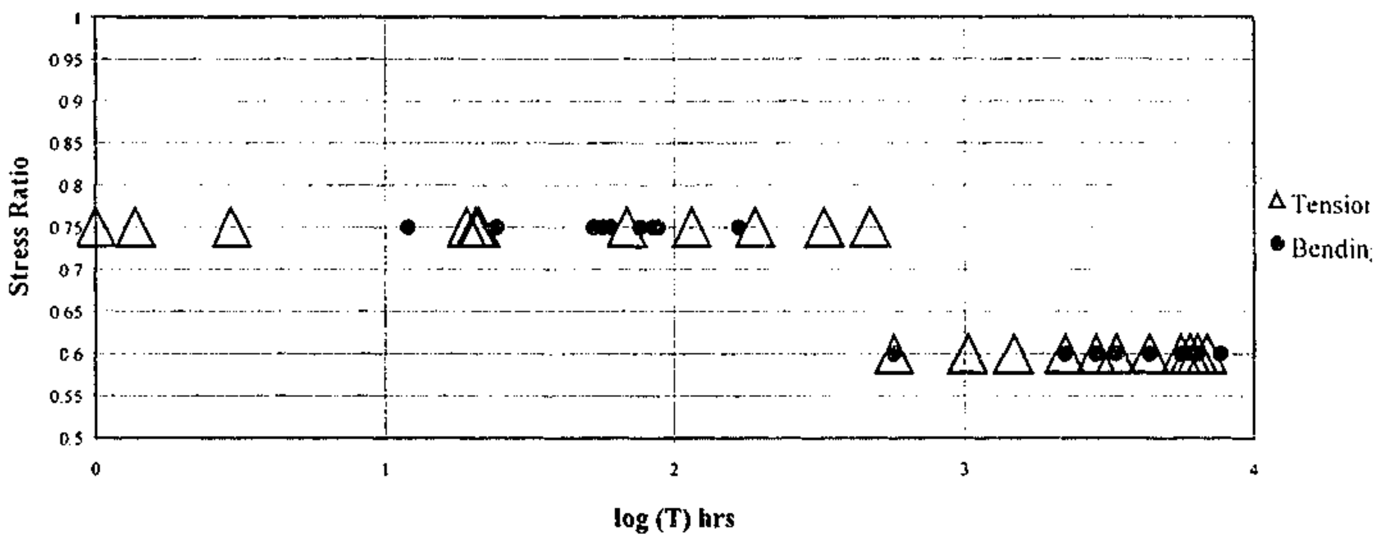


Figure 4.5 Elapsed Periods to Failure

The mean failure periods are summarized in Table 4.2 below.

Table 4.2 Mean Periods To Failure (Days) Indoor Situation

Loading	Stress Level*	
	0.75	0.60
Bending	3	189
Tension	3	201

* Stress Level is the ratio of applied stress to short-term ultimate strength.

The relative humidity/temperature chart drawn from the paper hygrograph plot is shown in Figure 4.6.

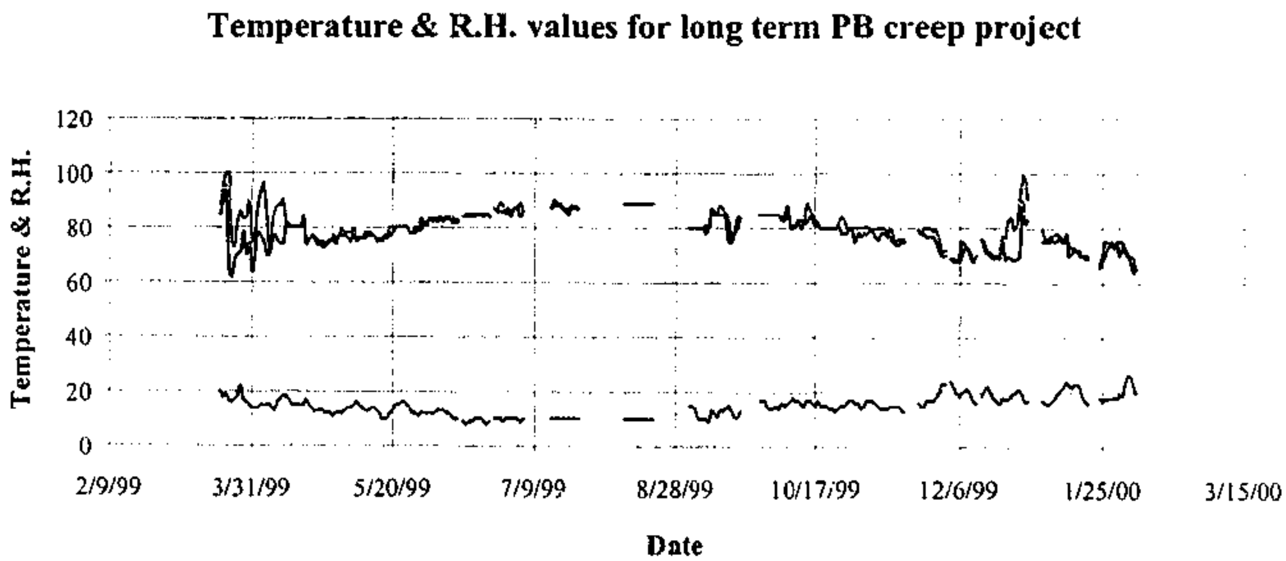


Figure 4.6 Charts of Relative Humidity and Temperature

4.4.5 Experiment 2

Aim.

To determine the elapsed period to failure and number of humidity cycles that occur when 19*mm* particleboard placed in natural ambient conditions in a sheltered external situation is loaded in bending and tension. Bending loads were 0.75, 0.60, and 0.48 of short-term modulus of rupture, and tensile loads 0.75, 0.60, and 0.48 of short-term ultimate strength.

Method.

Experiment 2 was conducted during two periods; specimens loaded to 0.75 and 0.60 load levels during the period 7 June 2001 to 13 July 2001, and to the 0.48 level during the period 31 August 2001 to 7 April 2002.

The method is identical to that of Experiment 1 except that the test rigs were situated under an uninsulated steel roofed shelter open on three sides to allow uninterrupted air flow to ensure that ambient external conditions obtained. Ambient humidity and temperature under the shelter were recorded continuously on electronic sensors. All specimens were conditioned to 65%*rh* at 20°*C* and their short-term ultimate strength determined as described in Experiment 1 before installation in loading rigs within the shelter on 1 November 2000. Fifty tension and thirty bending specimens with matched pairs were prepared as described for Experiment 1. On concluding Experiment 2a, ten tensile and ten bending specimens were similarly prepared and conditioned.

For tensile tests, twenty were loaded in to 0.75, twenty to 0.60, and ten to 0.48 of their individual short-term ultimate strength. For bending tests, ten were loaded to 0.75, ten to 0.60, and ten to 0.48 of their individual short-term modulus of rupture. Elapsed periods to failure were logged electronically for the 0.75 loading and manually for the 0.60 and 0.48 loadings.

Results

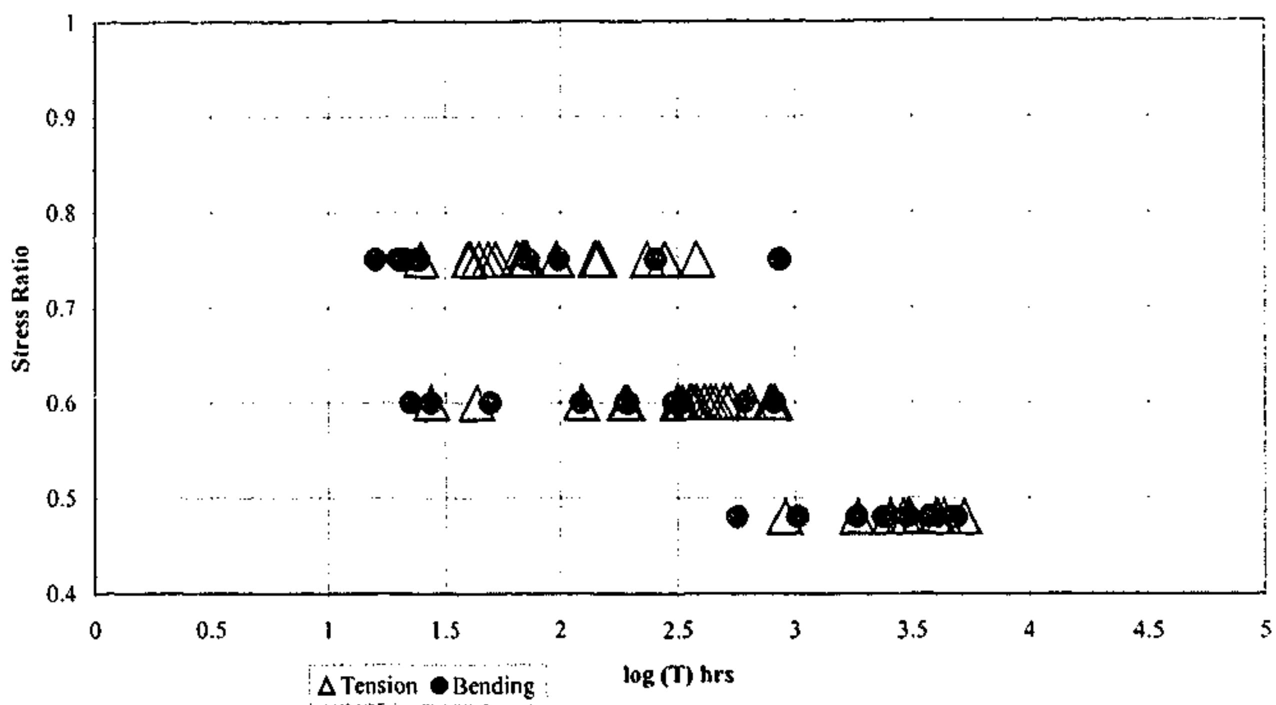


Figure 4.7 Elapsed Periods to Failure

Table 4.3 Mean Periods To Failure (Days) In Sheltered Outdoor Situation

Loading	Stress Level		
	0.75	0.60	0.48
Bending	3	10	120
Tension	5	15	122

* Stress Level is the ratio of applied stress to short-term ultimate strength.

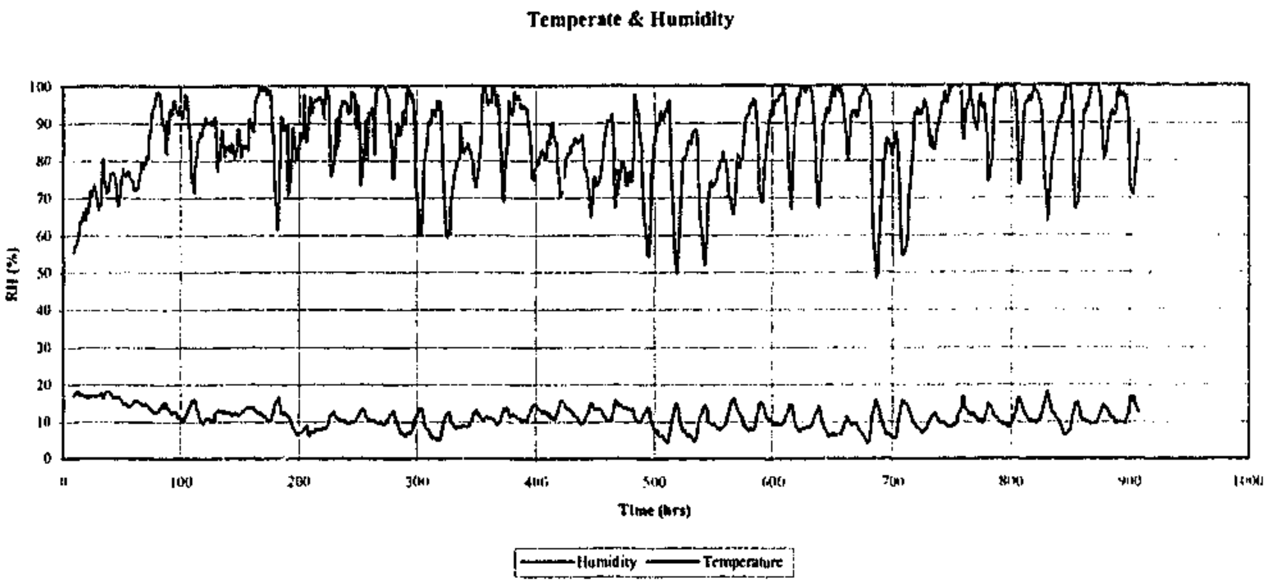


Figure 4.8 Relative Humidity and Temperature During 0.75& 0.60 Loading Period

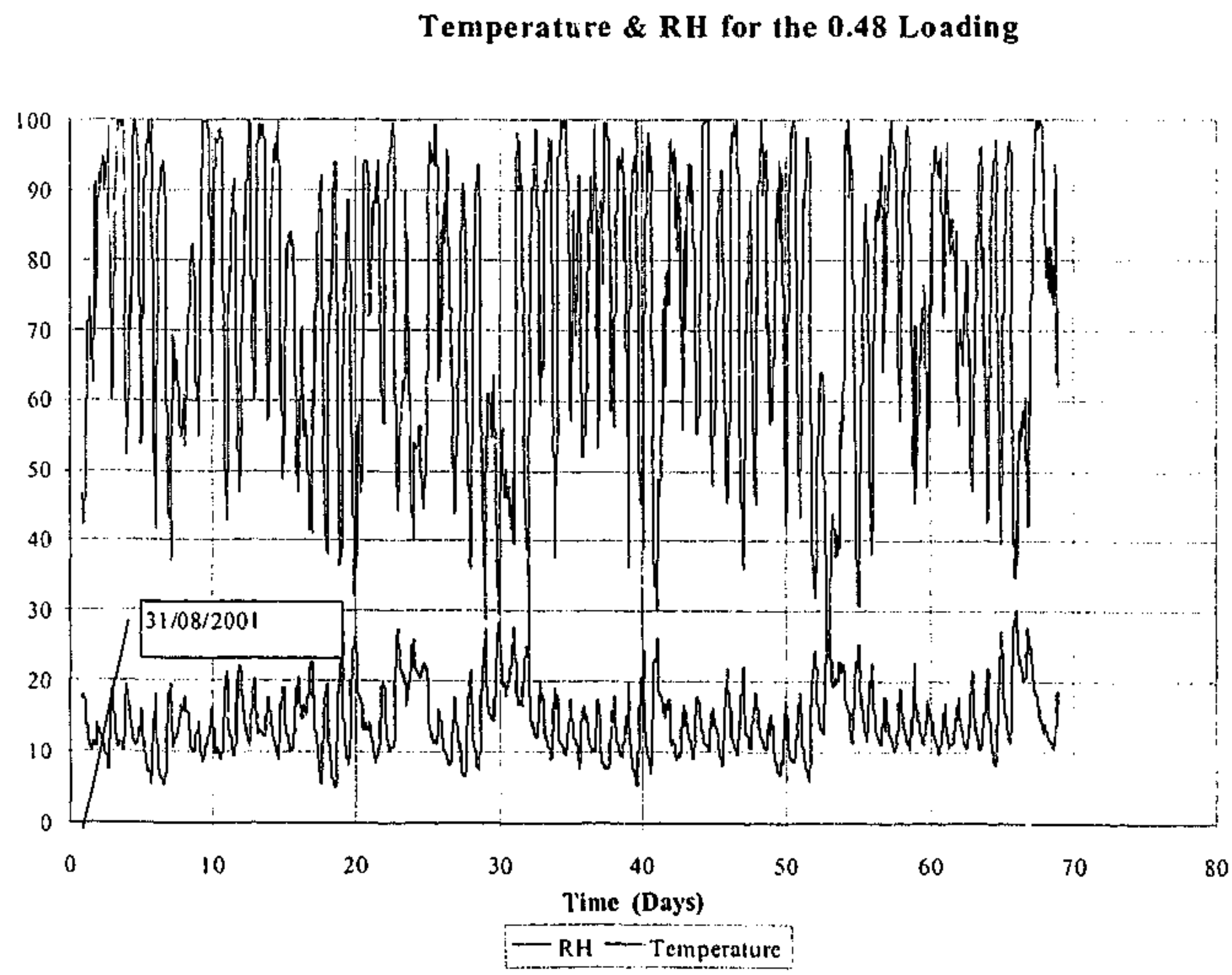


Figure 4.9 Relative Humidity and Temperature During 0.48 Loading Period

4.4.6 Experiment 3

Aim

To demonstrate that forced humidity changes that simulate large natural ambient humidity variations cause particleboard subjected to a high tensile stress to lose strength rapidly.

Method

Two matched pairs of specimens were prepared and conditioned for a minimum of three days at 20°C and 65%rh. After conditioning, the short-term ultimate tensile strength of one of each pair is measured. The two remaining specimens were placed sequentially in a small bench-top test chamber and subjected to a tensile load of 0.60 of their short-term ultimate strength while relative humidity was cycled from 65% to 90% to 65% over a 48hr period. The sawn edges of the first specimen were uncoated, the second specimen was coated with silicone over its sawn edges; its two (original board) faces remaining uncoated.

The humidity cycle consisted of four phases that were regulated to represent these conditions sinusoidually as could best be achieved with the available computer controlled air conditioning plant. The environment stabilized to 65%rh at 23°C prior to entering Phase A of the 48hr cycle. On entering Phase A, humidity was raised over 18 hours to 90%rh in eight uniform increments, held for 6 hours at 90%rh as Phase B, before being reduced by the same increments over the following 18 hours to 65%rh as Phase C. As the final Phase D, humidity was held at 65%rh for another 6 hours before entering the second 48hr cycle. The cycle was repeated with no change in load until failure occurred.

Results

The first specimen, uncoated, failed after 44 hours, virtually completing one cycle only. The second specimen, coated, failed after 104 hours, that is, after two cycles and 8 hours into Phase A of the third cycle. Two other coated specimens failed equally rapidly.

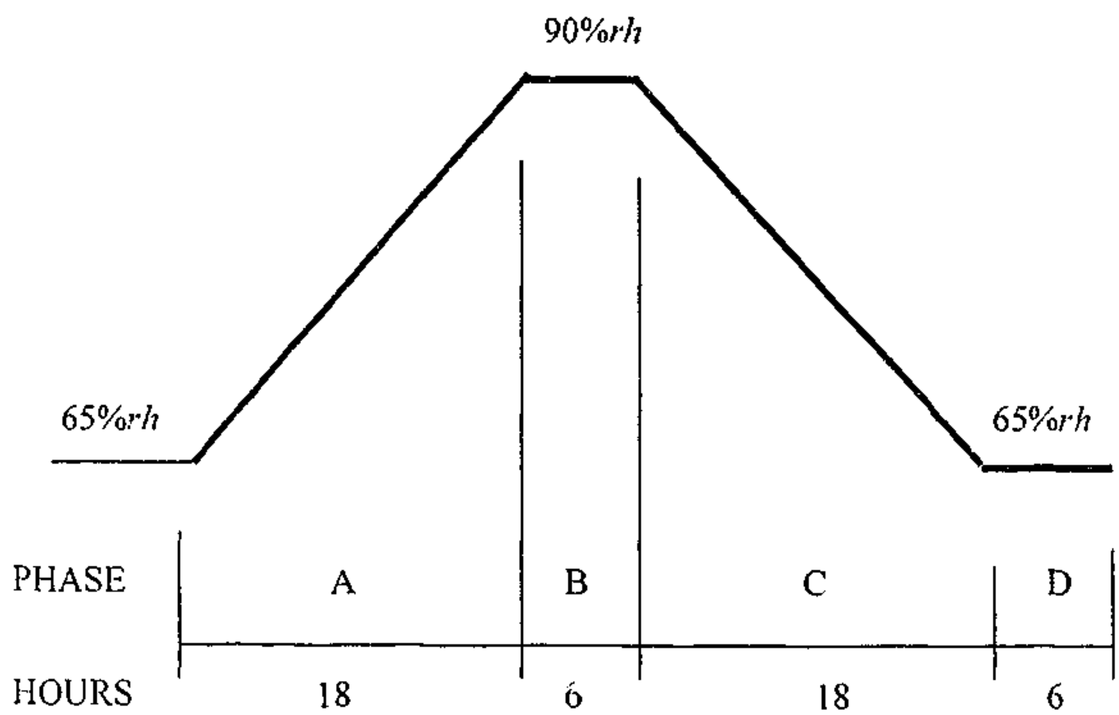


Figure 4.10 Programmed Relative Humidity 48hr Cycle

4.4.7 Experiment 4

Aims

To confirm the phenomena observed in Experiment 3 by placing, simultaneously, a number of specimens of particleboard under tension in a large environmental chamber programmed to deliver at $20°C$ a sinusoidal cyclic humidity regime of $65\%rh$ to $90\%rh$ to $65\%rh$ over $48hr$ periods.

To measure the elapsed period and number of humidity cycles that occur before failure of specimens loaded in tension to 0.48 of short-term ultimate strength whilst subject to this regime.

Method

Sixteen tension specimens were prepared and conditioned as earlier described. Short-term ultimate loads were obtained from matched pairs tested in one of the tension loading rigs used for Experiment 1. Following which the remaining eight specimens were placed in tension loading rigs within the large environmental chamber and loaded to 0.48 of their individual short-term ultimate strength on 30 November 2001. Following installation, the specimens were subjected to the forced humidity cycle used for Experiment 2 until all specimens failed. Humidity cycles were recorded on a paper hygrograph as for Experiment 1.

Results

All specimens broke within fifteen days, that is, within eight forced humidity cycles.

The large environmental chamber replicates the humidity cycles satisfactorily.

4.4.8 Experiment 5

Aim

To measure the elapsed period and number of humidity cycles that occur before failure of specimens of particleboard loaded in tension to 0.30 of short-term ultimate strength while subject at $20°C$ to forced humidity cycles of $65\%rh$ to $90\%rh$ to $65\%rh$ over $48hr$ periods.

Method

Twenty specimens with matched pairs were prepared and their individual short-term ultimate tensile strength determined as described in Experiment 1. They were then placed in tension rigs located in the large environmental chamber, loaded to 0.30 of short-term ultimate strength on 30 November 2000 and subjected to the forced humidity cycle used for Experiment 2. Elapsed periods to failure were recorded manually.

Results

- The first specimen failed, unexpectedly, by 14 February 2001. Investigation showed that the specimen was bound to the lower pin, consequently the rotation of the lever loading arm introduced a bending stress, which when added to the axial stress, was sufficient to cause the early failure.
- The remaining seven specimens reduced to six on 6 February 2002. All six remain intact at the time of writing, 31 August 2002, and will have been subject to nominally 250 cycles of humidity since installation in the chamber.
- The mean of the strain cycles remains constant, indicating that deformation has ceased.

4.4.9 Experiment 6

Aim

To measure the elapsed period and number of humidity cycles that occur before failure of specimens of P. radiata loaded in tension to 0.60 of short-term ultimate strength while subject at $20°C$ to forced humidity cycles of $65\%rh$ to $90\%rh$ to $65\%rh$ over $48hr$ periods.

Method

Twenty specimens with matched pairs were prepared and their individual short-term ultimate tensile strength determined as described in Experiment 1. They were then placed in tension rigs located in the large environmental chamber, loaded in tension to 0.60 of short-term ultimate strength on 1 November 2000 and subjected to the forced humidity cycle used for Experiment 2. Elapsed periods to failure were recorded manually.

Results

- The twenty original specimens reduced to eight by 20 July 2001, to six on 24 January 2002. Six remain intact at the time of writing, 31 August 2002, and will have been subject to nominally 250 cycles of humidity since installation in the chamber.
- The mean of the strain cycles remains constant, indicating that deformation has ceased.

4.4.10 Stress Relaxation – Period v Cyclic Moisture

It is believed by the writer that, at stress levels below approximately 60% of short-term ultimate strength, stress relaxation is governed by the number of moisture cycles as opposed to the passage of time. To provide a basis for comparison between the failure periods of the specimens subject to the forced cyclic humidity, specimens of particleboard and P. radiata were placed in tension rigs, identical to those used for the long-term tests, and located in a closed room in the Monash Caulfield structures laboratory under nominally steady conditions of $20°C$ and $65\%rh$.

A set of eight tension specimens of particleboard was installed and loaded on 9 June 1999 to 0.60 of short-term ultimate strength. Of the eight, two were uncoated and failed during the first and third weeks of August, the coated remainder of six were intact when withdrawn from test on 14 November 2000 to make way for P. radiata specimens. A set of eight tension specimens of P. radiata matched to the specimens placed under test in Experiment 6 was prepared and similarly loaded on 14 November 2000. Two were uncoated and failed within 4 weeks. Of the coated remainder of six, one failed c April 2001, and five were intact on 30 August 2002 when they were withdrawn from test. It is evident that silicone coatings applied to the cut edges of a specimen significantly, probably totally, reduce moisture sorption.

It appears that, although under high stress, mechanosorption arising from the small variations in moisture content occurring during these periods in an air-conditioned space is unable to cause significant loss of strength. Which supports the premise that stress relaxation is predominantly induced by large changes in moisture content rather the passage of time.

4.5 DISCUSSION

4.5.1 First Stage – Stress Relaxation in Natural Climate

4.5.1.1 Conclusions

Hygrographic records made during Experiments 1 and 2, (Figure 4.6), indicate that natural variations in relative humidity indoors reach a seasonal maximum of 25% and occur over 48*hour* periods in approximate sinusoidal cycles of 65%*rh* to 90%*rh* to 65%*rh*.

As observed during Experiment 1 at both the 0.75 and 0.60 stress level and the 0.48 level of Experiment 2, failures in bending and tension take place at practically the same rate, whether loaded normal to the surface or in-plane. No bending compression failures were visually detected, which indicates that failures are probably initiated by tensile, rather than compressive, stress and that long-term behaviour under tension can represent behaviour in bending.

The mean period to failure of the more highly stressed, (0.60 Level), specimens exceeds that of the lower stressed, (0.48 Level), specimens, the 50% or more difference being so great as to rule out any possibility that experimental error is significant. Which leads to the conclusion that the failure rate is independent of the loading period, and is, therefore, governed by the size and number of moisture changes experienced under load.

Experiments 1 and 2 indicate clearly that valid extrapolation for stress relaxation is only possible with consideration of the cumulative effects of moisture sorption and rates of moisture diffusion through the material during the test period. However, as the literature indicates, current stress relaxation models assume that loss of strength and creep is a logarithmic function of time and stress. The effects of ambient humidity variations are related to time, and the effects of moisture gradients, which exacerbate any localized weakening effect of stress concentrations within the material, is accounted for empirically by adjusting model parameters and coefficients, applied to material and changes in moisture content, to fit observed outcomes. The result is that no reliable extrapolation of stress relaxation is possible.

However, it appears that most of the creep, and therefore stress relaxation, will occur in twelve to fifteen months, which period includes several natural maximum changes in humidity and temperature and associated maximum changes in moisture content gradients. By inference, this means in structural design terms, that loss of strength and creep could be considered as uniform over that period, possibly reaching values not significantly less, although reached in a much shorter period, than their stable limits.

4.5.2 Second Stage – Stress relaxation in Forced Cyclic Humidity

4.5.2.1 Conclusions

Large cyclic humidity changes cause rapid failure of particleboard loaded to a level of stress that is too high to allow a mechano-sorptive limit state to develop.

Particleboard and defect free P. radiata under a tensile stress corresponding to 0.3 and 0.6 respectively of short-term ultimate strength, reach a stable limit state at which stress relaxation and deformation cease after being subjected at 20°C to 250 cycles of humidity that varies from 65%rh to 90%rh to 65%rh over 48hr periods.

While considerable care was taken to select defect free material, the initial failure rates in Experiments 5 and 6, particularly in the latter, draw attention to the weakening effect of moisture on stress concentrations, having regard to the stable limit state apparently reached by reconstituted timber and timber alike.

4.5.3 Review of Current Duration-of-Load Assumptions

The evidence that a stable limit state is apparently reached by reconstituted timber and timber alike raises important issues that require explanation. Many structural engineers, among them Hunt (1976) and Madsen (1992), believe that the "Madison" duration-of-load factor k_1=0.57 for structural grade timber under sustained load for 50 years is too high and thus unsafe.

The "Madison" factor, as presently specified by AS 1720.1-1997, applies to all structural timber irrespective of its situation, whereas, the limit state of 0.6 revealed by this study is for timber free of defects in sheltered outdoor situations. But, in fully exposed situations, changes in moisture content are double those that occur indoors, consequently mechanosorptive creep and stress relaxation is doubled. It is therefore concluded that a more probable duration-of-load factor for defect free timber in fully exposed situations is at least one stress grade (25%) lower at k_1=0.48. To be valid for defect free timber, this conclusion must be consistent with the limit state factor of 0.30 obtained for particleboard in sheltered outdoor situations. Consistency is demonstrated as follows.

It appears that, by chipping timber and reassembling the chips randomly as particleboard, the variability of the 5%ile strength of structural grade sawn timber is reduced from V_R=0.4 to V_R=0.1, assuming log normal distributions of strength for each. But the random reorientation of wood chips in particleboard reduces those acting in one direction by 50% and, thereby, its average strength by 50%. Hence, the nominal doubling of strength attained by randomly rearranging wood fibre and thereby removing the effects of defects, offsets, almost exactly, the 50% loss due to the higher variability of sawn timber caused by defects. Because mechanosorptive creep and relaxation is proportional to stress at levels normally encountered in structures, the doubling of stress in the effective 50% of particleboard fibres compared with the average stress across solid timber, doubles its mechanosorptive creep and relaxation. That is, in sheltered outdoor situations, the experimentally determined 50 year duration-of-load factor of 0.30 for particleboard is consistent with the inferred 50 year duration-of-load factor of 0.48 for structural grade sawn timber.

It is necessary that the reduction in strength caused by defects in structural grade sawn timber is accounted for consistently. AS 1720.1-1997, clause 2.4.1.2 *Effect on stiffness*, specifies that, for timber with moisture contents below 15%, the long-term deformation in bending and compression is double that in tension. At design stress levels, the elastic modulus in compression, E_C, equals that in tension, E_T, hence, under the same axial stress, compressive and tensile strains should be equal. Axial deformation is inversely proportional to E and cross sectional area, therefore, with E_C=E_T, the effective cross sectional area is halved under the action of compressive stress. It is apparent that the specified reduction of 50% in resisting compressive stress is due to the effects of defects on compression strength. For sawn timber generally, the presence of defects appears to lower the strength of defect free timber by roughly two stress grades, (56%), so that k_1 reduces to 0.38 in an SE2 environment.

The foregoing implies that dead load design stresses, as currently specified for sawn timber by AS 1720.1-(1997), could be around two stress grades too high in unsheltered external situations and, therefore, in sufficient error to lead to more frequent failures. But failures of timber structures are infrequent, and then due primarily to low grade material, faulty fabrication or poor joint design. Hence, some action(s) must be present to mitigate the implied reduction in strength and safety. A possible explanation is discussed below.

4.5.4 A Possible Explanation

Four mitigating actions seem to have effect. One rests with the geographical region in which a timber structure is situated. Taking the probable 50% dead load 'overstress' in SE3 as the basis, when referenced to the other environmental classifications, dead load design stresses could be around 25% high for an SE2 environment, of the correct order in an SE1 environment and possibly around 25% low for an arid region, (SE0). Geographical regions that embrace these sheltered external service environments in Australia are delineated in Fig. 3.2. Their boundaries indicate that the majority of timber structures are built in regions where SE1, or SE2 environments prevail, hence, relatively few would have members overstressed more than 25%.

A second action, similar to the first, is implicit in the halving of the annual deformation rate from 7% when fully exposed to weather (SE3) to 4% when situated indoors (SE2), (Kingston 1968). In the main, timber structures are protected from weather in a way that inhibits rapid moisture sorption and the moisture content of members may never reach equilibrium. They may, therefore, be regarded as situated in an SE1 or SE2 environment, and as such, subject to deformation and relaxation rates nominally half, or less, than those in unprotected timber exposed to weather. Hence, relatively few would have members overstressed more than 25%.

A third mitigating action results from the high proportion that live load generally forms of the total load supported by timber structures and the higher values assigned to design stresses as a consequence. The research data indicates that stress relaxation in timber is greater under permanent loads and lower under transient loads than is currently assumed. The duration-of-load factors, k_1, that modify live load design stresses as specified by AS 1720.1-1997 appear to under-estimate a structure's resistance to live loads by an amount that compensates for the over-estimated dead load resistance because, in certain conditions, not uncommonly met, lost strength and deformation are totally regained upon removal of the load.

Nakai and Grossman (1983), Grossman and Nakai (1987), demonstrated that when a load is applied intermittently to wood, with the period between successive applications not less than six times the period of application, the induced deformation virtually recovers completely. As any given loading period extends to occupy a greater proportion of the total time between successive applications, recovery reduces until the limiting case of ongoing deformation under a permanent load is reached. At that point, no recovery takes place while moisture content remains constant, although some recovery will occur under subsequent changes, particularly reductions, in moisture content. Interwoven into the effect is a more rapid recovery when moisture content changes after the load is removed, or alternatively, the greater incremental deformation experienced if it changes during the load's application. AS 1720.1-1997 recognises this phenomenon by allowing the reference to *"the creep component of deformation .... to be totally recovered"* when the period between load applications exceeds the loading period ten fold. The mechanosorptive component of creep is not referred to explicitly and is therefore implicit in *"the creep component of deformation"*.

The fourth action is tied to current design criteria for timber beams in which an unacceptable deformation is generally reached well before limiting stresses are created. This is because localized strength reducing defects have an average effect on elasticity over the length of a member whereas they may be the source of local failure in an otherwise sound member. In the case of framed structures, stress limits in principal members are more likely to be reached first, for example in trusses. Uncertainty in both design theory and construction methods increases the possibility that some members may be overstressed, but this is offset by the lower probability that an over-stressed member will be critically placed. Probabilistic aspects of design and construction are discussed in Chapter Five.

4.6 CONCLUSIONS

4.6.1 Postulate Verification

The evidence strongly suggests that the postulates have been verified and it is probable that the following conclusions are valid.

1. Under an average stress corresponding to 0.30 of short-term ultimate strength, 19*mm* particleboard, in a simulated sheltered external environment, reaches a stable limit state after which stress relaxation ceases. For limit states design

purposes, a conservative duration-of-load factor suitable for 19*mm* particleboard elements in structures in service for 50 years under sustained load in a sheltered external environment is k_1=0.3.

2. Under an average stress corresponding to 0.60 of short-term ultimate strength, 19*mm* thick defect free wood in a simulated sheltered external environment reaches a stable limit state after which stress relaxation ceases. For limit states design purposes, a conservative duration-of-load factor suitable for 19*mm* thick defect free wood elements in structures in service for 50 years under sustained load in a sheltered external environment is k_1=0.6.
3. Under a sustained stress in the order of that experienced by structures in service, the loss of strength in wood and particleboard is a function of the size and number of humidity induced moisture changes experienced.
4. The random chip orientation in particleboard imparts in-plane isotropy that increases resistance to compressive stress so that under sustained load, in-plane tension and compression strength are roughly equal and in bending, when loaded normal to the surface, failures do not necessarily originate in the compressive fibres.
5. Wood with defects fails more rapidly than wood free of defects when subject to ambient moisture changes under tensile stress in the order of that experienced by structures in service.

4.6.2 Efficacy of Experimental Method

A comparatively rapid method for determining stress relaxation factors for particleboard appears to be reasonably demonstrated. The consistent experimental results and simplicity of apparatus support the method's efficacy. In view of its rapidity, the method should have general application to any reconstituted wood product of any size

4.6.3 Structural Design Application

4.6.3.1 Duration-of-Load Factors for Live Load

The frequency and magnitude of moisture content changes during service, and the consequential recovery of strength and deformation between load applications, have yet to be fully brought into design procedures. Currently, for loading periods greater than 5 minutes duration, an increase in design stress is assessed according to the cumulative length of time under live load as obtained by simply adding together successive durations of live load application. Consequently, values of k_1 specified for live loads in AS1720.1-1997 appear overly conservative because they ignore the high proportion of total load formed by transient live load that satisfies the recovery condition of 1:6 loaded vs unloaded periods when placed on timber structures with their high strength/mass ratios.

Given this strength/deformation inter-relationship, it is highly probable that, in ambient conditions, strength and deformation fully recover between successive live loadings. More realistic classifications of live load, therefore, are needed so that temporary loads, which in fact are permanent, can be differentiated from true transient loads. For example, it is presently accepted that "movable" furniture forms around 40% of the live load in an office or a dwelling. Transient proportions of live load have been investigated for various types of building occupancy, but none of this work appears to have been done with the unique recovery characteristics of timber structures specifically in mind.

It may be argued therefore, that the duration-of-load factor specified in AS 1720.1-1997, Table 2.7, *Duration of load factors for strength*, for live load applied for around 5 days should increase by 50% from 0.77 to 1.00. This would offset the implied 50%

reduction that appears necessary to compensate for the lower dead load design stresses that are apparently necessary for structures in Service Class SE3. Increasing k_l by 50% for true transient live loads and decreasing it by 50% for dead loads results in nominally similar design stresses for combined dead and live loading, hence the same sized structural members still obtain for the majority of timber structures. Because the majority of structures incorporating timber components are located in an SE2 environment, it is conjectured that the recovery in strength and deformation that occurs in ambient conditions after the removal of live load may explain why failures due purely to overstress or excessive deformation, as distinct from poor design and construction methods, do not occur more often.

For loading periods less than 5 minutes duration, the mechanosorptive effect is unable to be activated in structural sized material and increases in k_1, as typically specified in AS 1720.1-1997 for such periods, are unaffected.

4.6.3.2 Duration-of-Load Factors for Dead Load

Because the long-term effect is a function of large moisture content changes, it is possible that these may occur when the dead load is first applied. Therefore, no increase is justified during this early period and the stable limit state factors, k_l=0.3 for particleboard and 0.6 for clear timber in an SE2 environment, should apply to all dead loading. From which, values of k_l for particleboard in other service environments and for structural timber are inferred and presented in Table 4.4.

Table 4.4 Inferred Duration-of-Load Factor, k_1, for Relaxation in Strength of Particleboard (680kg/m^3) and Initially Seasoned Timber, Defect Free (Clear) and Structural Grade, under Dead Load for 50 years.

Service Environment	Ambient %rh Ruling / Occasional	emc % Change		Duration-of-Load Factor, k_1 @ 20°C		
		P'board	Timber	Particle board	Clear Timber	Structural Timber
SE0	50 /.65	2	3	0.38	0.75	0.60
SE1	65 / 80	4	4	0.38	0.75	0.60
SE2	80 / 90	4	5	**0.30**	**0.60**	0.48
SE3	90 / 90+	5+	5+	*	0.48	0.38

Note: Ambient %rh refers to the maximum relative humidity that generally rules and to the exceptions that occur occasionally. * Particleboard is not suitable for use in exposed structures.

CHAPTER FIVE

THE RELIABILITY OF STRUCTURES WITH PARTICLEBOARD ELEMENTS

5.1 INTRODUCTION

Building regulations in Australia draw on relevant standard limit states design codes, and consequently depend on reliability theory for their implementation. An appreciation of the reliability theory supporting the Australian limit states timber structures design code, AS 1720.1-1997, is therefore a necessary precursor to the determination of appropriate reliability factors for structural particleboard elements and their connections.

Limit states design codes differ from working stress codes, in that, uncertainty is recognized explicitly and divided into two parts, that which accompanies estimates of action effects and that which is associated with estimates of structure resistance. The overall estimated reliability of a structure is expressed as a reliability index that reflects these uncertainties and attaches a theoretical probability of failure to the structure. The development of limit states design codes is currently based on the calibration of first-order probabilistic principles to working stress methodology.

The application of reliability theory to structures commenced with Cornell (1969) who proposed a limit states format for reinforced concrete design codes. Cornell saw structural design as the *"prediction through imperfect mathematical theories of the performance of structural systems constructed by fallible humans from material with variable properties when these systems are subjected to an unpredictable natural environment. All aspects of the problem are uncertain. The proposed format is more realistic because it is derived from a probabilistic model, the only kind of engineering representation which recognizes uncertainty and deals with it quantitatively and consistently."* Thirty years later, sufficient fundamental data on which to base true levels of reliability have yet to be collected, the sheer number of elements to be examined across a huge range of structures making the task virtually impossible. Consequently, the method is still subject to the type of criticism made by Gromala et al (1999) who recommended that writers of reliability based design specifications should *"encourage-----effort to narrow the gap between the mathematical precision of the method and the absence of information upon which to quantify key variables impacting insitu structural reliability."*

In Australian engineering practice, action effects are modified by load factors specified in AS 1170.1-1989, *SAA Loading Code, Part 1. Dead and Live Loads*, resistance estimates are modified by complementary capacity factors in AS 1720.1-1997, and materials are characterized by AS/NZS 4063-1992. In the following discussion, the effects of various sources of structural uncertainty are evaluated and their respective influence on resistance estimates for timber structures assessed. An anomaly between equations for structure resistance and material characterization was found to exist due to ambiguity in values of the capacity factor that is simultaneously applied to resistance estimates and material characterization in AS 1720.1-1997 and AS/NZS 4063-1992 respectively. No explicit account of design and construction variability is taken in AS 1720.1-1997. These short comings are investigated and as a result, the functions served by capacity factors in the design and construction process is clarified. A revised set of equations that removes the anomaly and accounts for variability in design and construction procedures also results.

No capacity factors specific to the use of structural particleboard are nominated by AS 1720.1-1997, hence, an evaluation of appropriate factors forms an integral part of this study.

5.2 THE RELIABILITY INDEX

5.2.1 Reliability

The converse of reliability, unreliability, is defined as the probability that a structure will fail. Failure will occur when either an ultimate limit state, such as partial or total collapse, overturning, sliding or uplift is exceeded, or a serviceability limit state when deflection or vibration reaches unacceptable levels or a want of durability exists. The limit state is reached when the safety margin equals or falls below zero, i.e., when $$R - S \leq 0 \quad (5.1a)$$

The probability of failure, p_F, can be stated mathematically as

$$p_F = p_r(R - S \leq 0) \quad (5.1b)$$

or $$p_F = p_r(R / S \leq 1) \quad (5.1c)$$

which is usually written in the form

$$p_F = \Phi(-\beta) \quad (5.1d)$$

where Φ is the cumulative frequency distribution of a unit normal variate, i.e., a standardized normal distribution with a mean of zero and a standard deviation of unity.

β is the Reliability Index.

Thereby, β is defined as a measure of the reliability of a given structure relative to the rate of failure per group of structures.

When the statistical distribution of R/S is known, it can be shown that the probability of failure to meet a specified strength limit state is approximated by

$$p_F \approx \Phi\left\{-\ln\left(\overline{R}/\overline{S}\right)\Big/\sqrt{V_R^2 + V_S^2}\right\} \quad (5.2)$$

where $\overline{R}$ is the mean resistance of the structure from computation or testing.

$\overline{S}$ mean load effect that the structure is designed to resist.

V_R coefficient of variation of the resistance system.

V_S coefficient of variation of the loading system.

By substitution in equation 5.2, the Reliability Index is given by

$$\beta = \ln\left(\overline{R}/\overline{S}\right)\Big/\sqrt{V_R^2 + V_S^2} \qquad 5.3a$$

Assuming that $\overline{R}$ and $\overline{S}$ have lognormal distributions and that variability in each is low, i.e. 30% or less, Ravindra et al (1969), made equation 5.3a linear with an error less than 10% within the range, $0.33 \leq \frac{V_R}{V_S} \leq 3$, so that

$$\beta = \ln\left(\overline{R}/\overline{S}\right)/0.75\left(V_R + V_S\right) \qquad 5.3b$$

giving

$$\overline{R}/\overline{S} = e^{0.75\beta(V_R+V_S)} \qquad 5.4a$$

or

$$\overline{R}e^{-0.75\beta V_R} = \overline{S}e^{0.75\beta V_S} \qquad 5.4b$$

Thus, the theoretical probability of failure of a structure, expressed as a Reliability Index, may be estimated from the ratio of its average resistance to the average design load effect and the coefficients of variation in each. Hence, β may be viewed as a simple means of comparing, theoretically, the relative reliability of structures in service.

The accuracy of the estimated probabilities of failure resulting from any comparison is governed by the accuracy of the available statistical data to characterize loads and resistance. In the absence of better data than that currently available, further refinement of the foregoing derivation of the Reliability Index would be inappropriate. Some values of the Reliability Index and related probability of failure are given in Table 5.1a.

Table 5.1a Values of Reliability Index β and probability of failure p_F. (Eq 5.3).

Reliability Index β	Probability of Failure p_F	
3.1	10^{-3}	1/1,000
3.7	10^{-4}	1/10,000
4.2	10^{-5}	1/100,000
4.7	10^{-6}	1/1,000,000

For building structures, the acceptable level of the probability of failure is very small, usually less than 1/10,000, (CSIRO 1974, Melchers 1987). Theoretical probabilities of failure are considered to range between $10^{-3.0}$ and $10^{-4.7}$. Within this range β can be approximated as

$$\beta = 1.2 - 0.6\log_{10}(p_F) \qquad 5.5$$

Values of the Reliability Index β and the probability of failure p_F from equation 5.5 are given in Table 5.1b.

Table 5.1b Values of Reliability Index β and Probability of Failure p_F. (eq 5.5).

Reliability Index β	Probability of failure p_F	
3.0	$10^{-3.0}$	1/1,000
3.5	$10^{-3.8}$	1/6,300
4.0	$10^{-4.7}$	1/50,000

5.2.2 Probability of Failure p_F.

5.2.2.1 Public Safety

The literature indicates that the assignment of true probability of in-service failure rates for normal building structures stands on limited evidence and, for unusual or complex structures, on even less evidence. The difficulty of relating probable and real failure rates stems from the very wide variety of structural forms and materials employed, the equally wide variety of design assumptions made as a matter of professional engineering judgement by individual designers, the standard of construction and the occurrence of rare and unforeseen events. Probabilistic theory is unable to cope with statistically unquantifiable effects as contained in the latter two in particular. It is apparent that, as the theoretical probability of failure reduces, the estimated failure rates become less realistic and cannot be taken as absolutes. Their meaning is only that failure is becoming more remote. Nonetheless, β is used in limit states design codes to establish theoretical levels of risk to life and disruption to welfare that are acceptable to the community at large. The following discussion is concerned with the levels of risk inherent in estimating strength limit states.

5.2.2.2 Acceptable Risk

The probability of failure for a given limit state, is based on experience, and is chosen to satisfy popular perceptions of acceptable risk. An appreciation of the risks due to human error is limited by meagre data, the subjective, psychological and sociological interpretations involved and the legal sanctions that inhibit third party viewing of most structural failures. The societal balance struck between economic and sociological imperatives underpins the level of acceptable risk and, although human error gives rise to most structural failures, no explicit provision for human error is made in Australian design codes. In Australia, concepts of risk now underpin all structural design codes, but have yet to be tested legally for the normal run of structural failures. Only when exceptional failures involving considerable loss of life or community disruption occur is the matter publicly aired through an open inquiry; some Australian examples being, the effects of Cyclone Tracey, the West Gate bridge collapse, and the Newcastle earthquake.

5.2.2.3 Social Criteria

One approach towards achieving the economical / sociological balance is given by Melchers (1987) who quoted the proposal by CIRIA (1977) that related the nominal probability of failure to the average number of people, N, in the vicinity of a failure during the design life, T years, of a structure and a social criterion μ. The target probability of failure may be represented by

$$p_F = \mu T / 10^4 N \tag{5.6}$$

The social criterion is concerned with a connection between probable loss of life and the probability of structural failure as distinct from the relative importance of a structure to overall community wellbeing. Values of p_F related to μ and corresponding values of β are given in Table 5.2.

Table 5.2 Target probability of failure related to Social criterion μ at T = 50 yrs (after CIRA-1977).

Nature of Structure	μ	N	p_F	β
Places of public assembly, dams	0.005	1000	2.5×10^{-8}	5.45
Domestic, office, trade, industry	0.05	100	2.5×10^{-6}	4.56
Bridges	0.5	10	2.5×10^{-4}	3.48
Towers, masts, offshore structures	5	1	2.5×10^{-2}	1.96

5.2.2.4 Warning of Failure

Melchers (1987) also quoted Allen (1981) who proposed another method to obtain the nominal target probability of structural failure with the expression

$$p_F = A\sqrt{T}/10^5 W \qquad 5.7$$

where A and W are activity and warning factors and T has the same meaning as above. Values of A and W that give target values of $p_F=7\times10^{-4}$ at 50 years are tabulated below.

Table 5.3 Target probability of failure v activity and warning factors at T=50 yrs. (After Allen 1981)

Activity factor	A	Warning factor	W	β	p_F
Post-disaster activity	0.3	No failure, fail-safe.	0.03	3.2	7×10^{-4}
Normal buildings	1.0	Gradual failure, warning likely	0.10	3.2	7×10^{-4}
Normal bridges	3.0	Gradual failure, view hidden	0.30	3.2	7×10^{-4}
Offshore structure	10	Sudden failure, no warning	1.00	3.2	7×10^{-4}

5.2.2.5 Importance of Structure

The choice of the reliability index is not specific to timber structures, but extends across the whole spectrum of structures, irrespective of materials and structural form, and reflects their importance in the maintenance of community well-being. Divergent opinion exists with regard to the selection of appropriate values of β for use with limit states design codes. For example, the Danish Code DS 413-1982 recognizes three classes of structural importance. The "Low Safety" class defined as negligible risk to life and small social consequence, the intermediate class as "Normal Safety" and the third "High Safety" class as great risk with large disruption; respectively β= 3.0, 3.5, and 4.0.

Pham and Kleeman (1999) widen the stratification of risk with the range of β proposed in Table 6.4. The indices relate a structure's importance to the level of risk associated with failure during a design life of 50 years. They also proposed that the tabulated relationships could be used to predict the risk for service periods other than 50 years. For example, the risk for a 100 year design life is obtained by raising structure importance one level, i.e., from Level 3 to Level 2, the risk for a 25 year life by dropping one level and for a 5 year life by dropping two levels. A revision of AS 1170.1-1992, under consideration presently, proposes the load effects risk levels in Table 5.4.

Table 5.4 Reliability Index, β, related to structure importance for 50yr design life. (After Pham and Kleeman 1999)

Risk Level	β	p_F	Structure Importance
1	*	*	Structures requiring special attention
2	3.9	0.48×10^{-4}	Structures with post disaster/essential service functions
3	3.7	1.08×10^{-4}	Structures presenting a high hazard to life or property
4	3.5	2.33×10^{-4}	Structures not otherwise classified
5	3.3	4.83×10^{-4}	Structures presenting a low hazard to life or property

5.2.3 Target Values of Reliability Index, β, Adopted

To indicate the relative importance of various structures to the community at large, as measured by the theoretical probability of failure, three strata of risk are currently targeted in Australian limit states design codes, namely, β=3.0, 3.5 and 4.0.

Reliability Index β=3.0.

The majority of housing is designed using simplified framing rules resulting in structures with standardized member sizes and connections which, as typically specified in AS 1684, *National timber framing code,* exhibit a theoretical probability of failure around 1/1,000. The members of the timber structures design code committee in Australia agreed that this risk of failure is evidently acceptable to the community and may be represented by a reliability index of β=3.0. Secondary timber structures generally, including house framing, are designed with the use of AS 1684 and constructed by highly competitive labour under minimal supervision, e.g., one inspection of a completed frame made by personnel with limited structural training is not uncommon.

Reliability Index β=3.5.

By contrast, as the importance or cost of a structure increases, so the quality of design and construction increases and the risk of failure decreases. To attain in primary timber structures a theoretical risk of failure equivalent to that obtaining in steel and concrete structures, which approaches 1/10,000, the committee endorsed the opinion held by Leicester (1980b) that it was necessary to raise β to 3.5. This latter index is suitable for primary components of structures where load sharing, which reduces the probability of failure, is absent. Single load path elements in house framing such as battens / purlins and isolated columns should fall into this strata of risk rather than β=3.0.

Reliability Index β=4.0.

Essential services structures and those performing post disaster functions, e.g., communication, transport, water supply and energy infrastructure, hospitals etc, are most likely to be rigorously designed with adequate testing and then constructed by experienced personnel adequately supervised. The reduced uncertainty results in a theoretical probability of failure around 1/50,000 represented by β=4.0.

The three indices align AS 1720.1-1997 with the range of β used in Australian limit states design codes for steel, concrete and masonry structures. The common values of β are thus theoretically capable of facilitating an interchange between various materials in a structure while retaining the target probability of failure. This interchange is predicated on the reliability factors that transform loads into load effects having common numerical values that are unrelated to materials so that resulting structural actions are also commonly assessed across all materials. The influence of load effects factors on estimates of structure resistance is discussed in the following section.

5.3 RELIABILITY FACTORS and CAPACITY FACTORS

5.3.1 Reliability Factors γ.

Reliability factors are used by the structural engineer to measure uncertainty in the behaviour of structures in-service. They account for the reliability of estimates of action effects and structure resistance. The uncertainty in estimates of action effects is concerned with potential overloads, physically uncertain dimensions, structural shapes and densities, and the analytical uncertainties introduced by simplistic assumptions regarding idealized load and stress distributions. The uncertainty in estimates of resistance derives from the inherent variability in component materials, design methodology, and construction procedures.

The reliability factors have values >1 and are applied during the design process through;

1. γ_S, which increases the estimated design action effects.
2. γ_R, which decreases the structure's estimated resistance to the design action effects.

The factor γ_R allows for design uncertainty, the quality of fabrication and erection, material / component properties, and the importance of a structure to community well-being. It is conveniently subdivided into

3. γ_B which decreases the estimated as-built resistance and
4. γ_M which decreases the estimated material / component property.
5. γ_I which increases reliability as the perceived importance of the structure grows.

In working stress design methodology, this can be stated mathematically as

$$\gamma_{0,WSD} = f(\gamma_S, \gamma_R) \qquad 5.8a$$

or

$$\gamma_{0,WSD} = f(\gamma_S, \gamma_M, \gamma_B, \gamma_I) \qquad 5.8b$$

where $\gamma_{0,WSD}$ is the in-service, or as-built, structure reliability factor. The complex function that produced $\gamma_{0,WSD}$ gave factors of safety that lay conventionally between 1.5 and 2.5. These developed from the engineering judgment accumulated during centuries of design and construction practice that generated, in the main, reliable 'fail-safe' structures in so far as they met the price the community was prepared to pay. However, it was accepted that there always remained some probability that failures due to overloads or unexpected behaviour could occur because so little was known about the occurrence of rare events.

For limit states design methodology, equation 5.8a takes the form of a simple product of the partial safety factors becomes

$$\gamma_{0,LSD} = \gamma_S \gamma_R \qquad 5.8c$$

or

$$\gamma_{0,LSD} = \gamma_S \gamma_M \gamma_B \gamma_I \qquad 5.8d$$

where $\gamma_{0,LSD}$ yields a specific value of the probability of failure, p_F, of a representative group of timber structures in-service. A calibration between working stress and limit states design methods, for a selected value of p_F, is thus effected by equalizing the respective values of γ_0. That is by putting

$$\gamma_S \gamma_M \gamma_B \gamma_I = f(\gamma_S, \gamma_M, \gamma_B, \gamma_I) \qquad 5.8e$$

from which

$$\gamma_{0,LSD} = \gamma_{0,WSD} \qquad 5.8f$$

5.3.2 Capacity Factors ϕ.

The strength reduction factor ϕ, called the capacity factor in structural design codes, is defined by AS 1720.1-1997 as *"a factor used to multiply the nominal capacity to obtain the design capacity"*, similarly, by AS 4100-1990, *Steel Structures*, and by AS 3600-1988, *Concrete structures code,* as *"a strength reduction factor"*, none with explanation of its basis or comment on its function.

The limit states condition is typically expressed as

$$R^* = R_{0.05} / \gamma_R = \gamma_S S_{0.95} \geq S^* \qquad 5.9a$$

where R^* is the estimated capacity of a structure to resist design action effects.

S^* estimated design action effect produced by the design loads.

$R_{0.05}$ 5%ile characteristic member / component design resistance.

$S_{0.95}$ 95%ile of the design action effect.

The capacity factor, ϕ, is the inverse of the resistance reliability factor, γ_R, so that equation 5.8a is written in limit states design codes as

$$R^* = \phi R_{0.05} = \gamma_S S_{0.95} \geq S^* \qquad 5.9b$$

As specified in AS 1720.1-1997, the limit states strength criterion

for member design is, $R^* = \phi R_{0.05} = \phi \Pi k\left(f_0' X\right) \geq S^*$ 5.10a

and for joint design is, $R^* = \phi N_j = \phi \Pi k\left(n Q_{k,j}\right) \geq S^*$ 5.10b

where f_0' the 5%ile characteristic stress appropriate to the failure mode.

Πk strength factors accounting for variability in environment and structure / component configuration.

X geometric property appropriate to the failure mode.

N_j characteristic joint strength.

Q_k 5%ile characteristic connector strength.

n number of connectors.

Limit states design methods enable the relative probability of failure of various structures to be measured by the Reliability Index. In AS 1720.1-1997, ϕ has three values, to suit three levels of rcliability which are dependent on levels of structure importance defined by their probability of failure. The independent role that the

Reliability Index plays in structural design is presently obscured. Its function is clarified in the following section.

In timber structures, elements are primarily rectangular. Consequently, real shapes and sizes are used to obtain sectional properties. For practical purposes the effect of geometric variability on the value of X is nil and, thus, has no effect on ϕ. Apart from the effect of the duration-of-load factor, k_1, on the calibration of long term strength to short-term characteristic strength, the product of the other modification factors, Πk, has no influence on ϕ.

The ensuing discussion considers the effects that control structure reliability as exercised through the Reliability Index and the capacity factor ϕ.

5.4 CALIBRATION PROCEDURES

5.4.1 Introduction

First published in 1972 as CA 65, *SAA Timber engineering code,* the Australian Standard AS 1720-1988, *Use of timber in structures,* was the final Australian working stress timber design code published. It has since been published in limit states format as AS 1720.1-1997 and referenced in the Building Code of Australia. To be acceptable to the building industry, it was necessary to ensure that AS 1720.1-1997 produced reliability outcomes consistent with its working stress predecessors via a suitable calibration. The calibration must therefore satisfy equation 5.8f by choosing appropriate values of ϕ (or its inverse γ_R) and γ_S. Because the process is exact for one particular load combination only, it is only possible to satisfy equation 5.8f approximately for other combinations.

During the calibration, a comparison with the theoretical reliability of steel and concrete structures revealed that primary structures designed to AS 1720-1988 and constructed with sawn timber were likely to be under-designed for strength by some 25%. Fortunately, because the serviceability limit state generally governs the design of timber structures, the implicit higher probability of failure revealed was infrequently brought into question. The calibration indicated that the theoretical reliability index for timber structures was around 3.0 for strength limit states rather than the target 3.5 aimed at by structures designed to comply with limit states steel and concrete design codes. The opportunity presented by the conversion of AS 1720-1988 to a limit states format was taken to unify the theoretical reliability expressed by all structural codes.

The application of reliability theory to timber structures in Australia was developed essentially by Leicester (1974, 1980, 1980a, 1980b) who effected a reasonable compatibility between AS 1720.1-1988 and AS 1720.1-1997 and raised the reliability timber structures to match that of steel, concrete and masonry structures. The following discussion demonstrates how this was done.

5.4.2 Assessment of β drawn from Working Stress Design Methods

For working stress design methods, characteristic material properties were based on standard 3-5 minute tests at 20°C and 65%rh with defects located in the worst position. The test data were converted to the basic working stresses specified in CA 65 and AS 1720.1-1988 by factors correlated to the standardised visual stress grading of scantling sized timber.

To obtain basic stresses, the characteristic properties of small clear specimens were reduced by a materials reliability factor, γ_M, to account for defects in the timber as distinct from variability in the wood substance, and for the variability introduced by sampling procedures and test methods. For example, the characteristic strength of a member under bending was $\gamma_{M,bending} = 2.22 / GF$, under tension, $\gamma_{M,tension} = 0.5\gamma_{M,bending}$, and under compression, $\gamma_{M,compression} = 0.75\gamma_{M,bending}$. The grade factor ($GF$) is the ratio of the characteristic bending strength of a particular structural grade of scantling to that of small clear specimens, i.e., Structural Grade 1 - $GF = 0.75$, St. Gde. 2 - $GF = 0.6$, St. Gde. 3 - $GF = 0.48$, St. Gde. 4 - $GF = 0.375$.

As Leicester (1974) explained, "*the material factor $\gamma_{(M)}$ contains not only an allowance for material variability but also allowances for variability in the effects of duration of load effects, the effects of degrade, the effects of grading procedures and the effects of workmanship,*" the latter being "*particularly important in the fabrication of joints.*" Thus, in working stress design methodology, some aspects of γ_R and γ_I are embraced by γ_M in the complex function in equation 5.8a.

To facilitate the transition to limit states design, Leicester (1980b) based a calibration procedure for the strength of scantling timber on the correlation between standardized visual stress graded and machine stress graded Pinus radiata, it being the only structural timber then being mechanically graded.

Its working strength in a structure, R_{BASIC}, determined for a fifty year load duration, was given by

$$R_{BASIC} = \frac{k_1}{\gamma_R} R_{0.025} \qquad 5.11a$$

where $R_{0.025}$ was the 2.5%ile strength of the population of machine graded timber when tested with defects located to cause the worst effect.

Substituting in equation 5.11a, $k_1 = 9/16$ to account for the reduction in strength over 50 years, and putting $1/\gamma_R = 0.8$ to account for the "*small accidental overloads, errors in design assumptions, and defects in workmanship*" referred to by Pearson (CSIRO 1958)

gives $$R_{BASIC} = R_{0.025}\left(\frac{9}{16}\right)0.8 \qquad 5.11b$$

When multiplied by 1.75, (nominally $16/9$), to return the 50 year prediction to a short term ultimate strength, (5 minute standard beam test), and divided by a statistically determined defects dispersion factor of $1/1.15$, (Leicester 1980), to allow the worst defect(s) to be randomly located, the estimated structure resistance became

$$R^* = R_{BASIC}\, 1.75/1.15 \qquad 5.11c$$

By substitution in equation 5.11b

$$R_{0.025} = 1.44R^{*} \qquad 5.12a$$

Leicester (1980a) assumed the distributions of R^{*} and $R_{0.025}$ to be lognormal, and by taking a coefficient of variation V_R of 0.40 as typical for structures built from mechanically stress graded sawn timber, related the 2.5%ile strength to mean strength concluding that

$$R_{0.025} = 0.435\overline{R} \qquad 5.12b$$

Substituting equation 5.12a in 5.12b gives

$$\overline{R} = 3.32R^{*} \qquad 5.13a$$

or

$$R^{*} = 0.3\overline{R} \qquad 5.13b$$

Given the criterion $R^{*} = S^{*}$ equation 5.13a may be written

$$\frac{\overline{R}}{\overline{S}} = 3.32\frac{S^{*}}{\overline{S}} \qquad 5.14$$

By substituting equation 5.14 into equation 5.3b, Leicester (1980a) obtained the reliability index in terms of load effects only as

$$\beta = \ln 3.32\left(\frac{S^{*}}{\overline{S}}\right) \Big/ 0.75(0.4 + V_S) \qquad 5.15$$

To compare the reliability of structures subject to dead, wind and live loads, Leicester (1980a) substituted typical values for V_S in equation 5.15 obtaining values of β for each as given in Table 5.5.

Table 5.5 Reliability Index β of structures with $V_R = 0.4$ for various load effects.

Load Effect	V_S	$S^*/\overline{S}$	β
Live load	0.30	1.4	2.9
Wind load	0.20	1.2	3.1
Dead load	0.10	1.0	3.2

From this comparison, Leicester (1980b) concluded *"that a safety index $\beta = 3.0$ is appropriate for the current usage of timber. This may be related to a usage that is predominantly concerned with secondary structures. However, for major structures a safety index of $\beta = 3.5$ may be more appropriate"*. The safety, or reliability index, β, thus provides the link that enables working stress design methods to be calibrated to limit states methods.

5.4.3 Calibration of Reliability Index, β, to Limit States Design Methods

To attain the three target values of β = 3.0, 3.5 and 4.0, it was necessary to evaluate the relative effect of the reliability factors in equation 5.8b. Leicester (1980a) dealt with this as follows.

The resistance side of the safety index equation, 5.4b, may be written

$$R e^{-0.75\beta V_R} = \frac{R_{0.05}}{\gamma_R} \qquad 5.16$$

Taking the resistance terms from equation 5.9a, and substituting for R^* in equation 5.16 gives

$$\gamma_R = \left(R_{0.05} / \overline{R}\right) e^{0.75\beta V_R} \qquad 5.17a$$

hence $$\phi = \left(\overline{R}/R_{0.05}\right)e^{-0.75\beta V_R} \qquad 5.17b$$

The load or action effects side of equation 5.4b may be expressed as

$$\overline{S}e^{0.75\beta V_S} = \gamma_S S^* \qquad 5.18a$$

hence $$\gamma_S = \left(\overline{S}/S^*\right)e^{0.75\beta V_S} \qquad 5.18b$$

Substituting in equation 5.18b typical values of V_S and $\frac{S^*}{\overline{S}}$ from Table 5.5, Leicester (1980b) obtained values of γ_S in terms of β as given in Table 5.6.

Table 5.6 Load effects factor γ_S related to β and load effects. (equ. 5.18).

Load Effect	γ_S		
	β=3.0	β=3.5	β=4.0
Live load	1.41	1.58	1.76
Wind load	1.31	1.41	1.52
Dead load	1.25	1.30	1.35
Mean values	1.32	1.43	1.54
Rounded values	1.3	1.4	1.5

The nominal "Rounded values" of γ_S in the ultimate row of Table 5.6 were adopted as typical reliability factors for load effects.

Since for a limit state $R^* = S^*$, substituting 5.17a in 5.8b gives

$$\gamma_0 = \gamma_S \left(R_{0.05} / \overline{R}\right) e^{0.75 \beta V_R} \qquad 5.19$$

Hence, by calibrating γ_R against the rounded values of γ_S, Leicester (1980a) obtained γ_0 in terms of V_R and β. The computation of γ_0 was simplified, Leicester (1980b), by setting up the empirical linear equations 5.20 that closely approximate equation 5.19.

For $\beta = 3.0$ $$\gamma_{0,3.0} = 1.3 + 0.7V_R \qquad 5.20a$$

For $\beta = 3.5$ $$\gamma_{0,3.5} = 1.4 + 1.5V_R \qquad 5.20b$$

For $\beta = 4.0$ $$\gamma_{0,4.0} = 1.5 + 2.5V_R \qquad 5.20c$$

Values of γ_0 from equations 5.19 and 5.20 are summarised in Table 5.7 as $\gamma_{0,3.0}$, $\gamma_{0,3.5}$, and $\gamma_{0,4.0}$ for the three target values of β=3.0, 3.5, and 4.0 respectively. Values from equation 5.19 are in italics.

Table 5.7 Structure reliability factor, γ_0, related to resistance variability, V_R, reliability index β and p_F.

V_R	β=3.0, p_F=$10^{-3.0}$		β=3.5, p_F=$10^{-3.8}$		β=4.0, p_F=$10^{-4.7}$	
	$\gamma_{0,3.0}$		$\gamma_{0,3.5}$		$\gamma_{0,4.0}$	
0.1	*1.38*	1.37	*1.54*	1.55	*1.71*	1.75
0.2	*1.44*	1.44	*1.68*	1.70	*1.94*	2.00
0.3	*1.51*	1.51	*1.82*	1.85	*2.18*	2.25
0.4	*1.58*	1.58	*1.97*	2.00	*2.45*	2.50

The foregoing process related a structure's theoretical reliability to the theoretical probability of failure and the variability in its estimated as-built resistance. It will be noted that when V_R=0.4, a 58% increase in γ_0 is obtained by increasing β from 3.0 to 4.0, i.e., by either using material two stress grades higher in strength or reducing design stresses by two stress grades (56%). Similarly, when V_R=0.1, the same increase in reliability results by raising γ_0 around one stress grade (25%).

The efficacy of Leicester's calibration may be judged by comparing working stress factors of safety with limit states reliability factors. From Table 5.7 it can be seen that for $\beta = 3.5$ (primary structures or primary members in normal non-redundant structures) the factor of safety varies between 2.0 and 1.55 when V_R varies between 0.4 and 0.1 which is consistent with normal working stress design factors of 1.5 for reliable structures and 2.0 or more for less reliable structures. Similarly, for secondary members or redundant structures where the reduced reliability of $\beta = 3.0$ obtains, respective working stress safety factors reduce to around 1.6 and 1.4 respectively.

5.4.4 Evaluating the Capacity Factor ϕ

Referring to Ravindra and Galambos (1978), Leicester (1986) expressed R^* and S^* in terms of their respective reliabilities and introduced another factor k_{COM}, known as the 'committee factor,' that reflects the degree of confidence held by a Code Committee in estimates of ϕ and γ_S.

It is both subjective and arbitrary and, as confidence improves, k_{COM} approaches unity and, consequently, the coefficients of variation of ϕ and γ_S, respectively V_R and V_S, tend to zero.

To introduce k_{COM}, equations 5.17b and 5.19 become

$$\phi = k_{COM}\left(\overline{R} / R_{0.05}\right)e^{-0.75\beta V_R} \qquad 5.21$$

and

$$\gamma_S = k_{COM}\left(R_{0.05} / \overline{R}\right)e^{0.75\beta V_S} \qquad 5.22$$

AS 1720.1-1997 assumes $k_{COM} = 1.0$, and values of ϕ may be related to reliability indices of 3.0, 3.5 and 4.0 by substitution in equation 5.17b. That is, when

$\beta = 3.0$ $$\phi_{3.0} = \left(\overline{R} / R_{0.05}\right)e^{-2.25V_R} \qquad 5.23a$$

$\beta = 3.5$ $$\phi_{3.5} = \left(\overline{R} / R_{0.05}\right)e^{-2.625V_R} \qquad 5.23b$$

$\beta = 4.0$ $$\phi_{4.0} = \left(\overline{R} / R_{0.05}\right)e^{-3V_R} \qquad 5.23c$$

Substituting in equations 5.23 relevant values of V_R drawn from Table 5.8, and values of $\overline{R} / R_{0.05}$ for lognormal distributions from Table 5.27, the values of $\phi_{3.0}$, $\phi_{3.5}$, and $\phi_{4.0}$ given in Table 5.9 are obtained.

Table 5.8 Nominal Property Variations, V_R, obtained from Tests at $20°C$ - $65\%rh$.

Component	V_R
Particleboard	0.10
Laminated veneer lumber	0.10
Plywood	0.20
Laminated timber	0.20
MGP and A17 grade timber	0.30
Sawn timber seasoned	0.40
Timber trusses	0.20
Nail and nail plated joints	0.15
Finger joints	0.20
Bolted and ring fastened joints	0.30

Table 5.9 Capacity Factors ϕ related to Component Variability V_R. (equ. 5.23).

V_R	$\phi_{3.0}$	$\phi_{3.5}$	$\phi_{4.0}$
0.10	0.94	0.91	0.87
0.20	0.90	0.83	0.78
0.30	0.86	0.77	0.69
0.40	0.83	0.71	0.61

Table 5.10 Values of the Capacity Factor, ϕ, taken from Table 2.5, AS1720.1-1997.

Component	Component Property V_R	Secondary Structure $\phi_{3.0}$	Primary Structure $\phi_{3.5}$	Essential Services $\phi_{4.0}$
Timber, visual grade, poles.	>0.40	0.80 *0.80*	0.65 *0.65*	0.60 *0.60*
Timber, proof or machine grade.	0.30	0.85 *0.85*	0.70 *0.75*	0.65 *0.70*
Glulam.	0.15-0.20	0.85 *0.90*	0.70 *0.85*	0.65 *0.80*
Timber, MGP grades,	0.30-0.40	0.90 *0.85*	0.75 *0.70*	0.70 *0.65*
Plywood	0.15-0.20	0.90 *0.90*	0.80 *0.80*	0.75 *0.75*
LVL	0.08-0.12	0.90 *0.95*	0.85 *0.90*	0.80 *0.85*
Screws, nails/plates joints	≤0.20	0.85 *0.90*	0.70 *0.80*	0.70 *0.75*
Bolts, ring fastener joints	>0.30	0.75	0.65	0.60
Particleboard	0.04-0.10	*0.95*	*0.90*	*0.85*

Note: Rounded values of ϕ (in italics) drawn from Table 5.9.

Table 5.10 is an extract from AS 1720-1997, Table 2.5 *Values of capacity factor (ϕ) for calculating the design capacity (ϕR) of a structural member appropriate to the type of structural material and application of structural member.* A comparison of Tables 5.9 and 5.10 reveals that the capacity factors specified in AS1720.1-1997 were derived from equations 5.23, and except for some minor, industry generated anomalies, (MGP factors appear too high, glulam factors too low), the two tables are consistent. For particleboard, which has characteristic property coefficients of variation less than 0.1, rounded ϕ values suitable for structural design are 0.95, 0.9 and 0.85 for β=3.0, 3.5 and 4.0 respectively.

Values of capacity factors in the Danish Code DS 413-1982 are arranged to suit three classes of structure importance and two of material quality. For quality controlled material, ϕ is 0.81, 0.74 and 0.67 respectively and for uncontrolled material, 0.74, 0.67 and 0.60 respectively. The factors correspond generally to the values of ϕ in Table 5.10, the former set for components with V_R around 0.3, the latter set for materials with V_R in excess of 0.4.

5.4.5 Typical Load Effects Parameter, $\gamma_{S,typ}$.

The limit states loading code, AS 1170.1-1989, specifies a number of load combinations that are variously factored to provide the load effect particular to any one combination. Therefore, to facilitate an equalization of outcomes between limit states and working stress design methods, a typical parameter to represent the variety of possible combinations of factors is needed. Although not explained in the literature, Leicester (1980b) assumed a typical parameter, $\gamma_{S,typ}$, of 1.35. Reference to Table 5.6 reveals that this value was interpolated between mean values of γ_S for secondary and primary structures, i.e., between γ_S=1.32 and 1.43 for reliability indices of β=3.0 and 3.5 respectively. The parameter is applied to 5%ile characteristic properties determined by AS/NZS 4063-1992. Its value is further discussed in para 5.5.3d.

5.4.6 Calibration of Design Resistance Outcomes

AS/NZS 4063-1992 specifies that, for a sample of size n with 5%ile strength of $R_{0.05}$, an estimate of the 5%ile properties of a population determined with 75% confidence is given by

$$R_{k,0.05true} = \left[1 - \left(2.7V_M / \sqrt{n}\right)\right] R_{0.05,sample} \qquad 5.24$$

Derivation of the materials reliability factor,

$$\gamma_M = 1 - \left(2.7V_M / \sqrt{n}\right) \qquad 5.25$$

due to Leicester (1986), accounts for uncertainty in the true characteristic value because the sampling method may provide strength data that is not truly representative of the population. V_M is the variability of the sample data.

For working stress design, AS/NZS 4063-1992 specifies characteristic basic properties as

$$R_{BASIC} = R_{k,0.05true} / 1.75(1.3 + 0.7V_M) \qquad 5.26a$$

Hence, the characteristic 5%ile stress appropriate to a particular mode of failure, f_{BASIC}, is written

$$f_{BASIC} = f_{0.05,true} / 1.75(1.3 + 0.7V_M) \qquad 5.26b$$

Equation 5.26b is the basic stress cited in AS 1720.1-1988, which, expressed in terms of the complex function $f(\gamma_S, \gamma_M, \gamma_R, \gamma_I)$, (equation 5.8b), becomes

$$f_{0.05,true} = k_1 \gamma_{0,3.0} f_{BASIC} \qquad 5.27$$

Therefore, given that $\gamma_M, \gamma_S, \gamma_I$, are each unity, the overall reliability factor, γ_R, for a structure in service with a reliability index of 3.0 is $\gamma_{0,3.0} = 1.3 + 0.7V_R$, (equation 5.20a). The 50 year duration-of-load factor, $k_1 = 1.75$.

For limit states design, normalised 5%ile characteristic properties, $R_{k,norm}$, are given by AS/NZS 4063-1992 as

$$R_{k,norm} = 1.35 R_{k,0.05true} / \phi(1.3 + 0.7V_R) \qquad 5.28a$$

Hence, the characteristic 5%ile stress appropriate to a particular mode of failure, f_o', is written

$$f_0' = 1.35 f_{0.05,true} / \phi(1.3 + 0.7V_R) \qquad 5.28b$$

which is the characteristic stress cited in AS 1720.1-1997, Table 2.4.

Expressed in terms of reliability factors, $\gamma_S \gamma_M \gamma_R \gamma_I$, equation 5.28b becomes

$$f_0' = \frac{\gamma_{S,typ} \gamma_R f_{0.05,true}}{\gamma_{0,3.0}} \qquad 5.29a$$

or

$$f_{0.05,true} = \frac{\gamma_{0,3.0} \phi f_0'}{\gamma_{S,typ}} \qquad 5.29b$$

In equation 5.29b, $\gamma_{0,3.0} = 1.3 + 0.7V_R$, γ_I is embraced by ϕ, and γ_M is part of f_0'. AS/NZS 4063-1992 uses 1.35 as the value of $\gamma_{S,typ}$, and defines ϕ = "*the capacity factor to be applied in the design code*", AS 1720.1–1997.

An equivalent design resistance outcome obtains when $\gamma_{0,LSD} = \gamma_{0,WSD}$, (equation 5.8f). Hence, a theoretically equivalent design resistance outcome obtains when the respective characteristic properties, $f_{0.05,true}$, as evaluated for limit states and working stress use, are equated. Thus

$$f_0' = \frac{k_1 \gamma_{S,typ}}{\phi} f_{BASIC} \qquad 5.30a$$

giving

$$f_0' = \frac{2.36}{\phi} f_{BASIC} \qquad 5.30b$$

To produce a conversion factor of 2.95 it is necessary to put ϕ=0.8.

Hence, for member design using AS1720.1-1997, the calibration is given by

$$f'_o = 2.95 f_{BASIC} \tag{5.30c}$$

and for joint design using AS1720.1-1997, the calibration equation specified in AS 1649-1998, *Timber-Methods of test for mechanical fasteners and connectors-Basic working loads and characteristic strengths*, for laterally loaded nails and screws is

$$Q_k = 2.95 Q_{BASIC} \tag{5.30d}$$

where Q_k is the characteristic limit state fastener design load.

Q_{BASIC} the working stress fastener design load.

All basic stresses, specified in AS1720.1-1988 for the design of members, and laterally loaded nailed or screwed joints, were multiplied by 2.95 to give the corresponding 5%ile characteristic design strength specified in AS1720.1-1997.

Because the calibration factor depends on $\gamma_{S,op}$, it is subject to change as better statistical data on load effects is gained. Unfortunately, the adjustments to R_k, Q_k and ϕ made necessary by any change in the value of $\gamma_{S,op}$ would also require simultaneous amendments to AS/NZS 4063-1992 and AS 1649-1998.

5.4.7 Discussion

The choice of $\gamma_{S,op}$, γ_R (=1/ϕ), and β (γ_I), obviously affects the design of house framing and thereby the economic viability of the sawn timber industry in Australia. It was therefore of the utmost importance that the structural design of highly redundant domestic structures that had a theoretical reliability index of $\beta = 3.0$ when made from graded timber with a variability of 0.4 would match, whether estimated by working stress or limit states methods. CSIRO, with considerable industry involvement over a lengthy period, selected a wide field of data points to compare framing designs by computed by either method. The comparisons were treated conservatively until consensus was reached that the application of reliability theory yielded results sensibly consistent with those obtained by working stress methods as expressed in AS 1684-1999, *Residential timber framed construction.* In the evolution of the capacity factor, $\gamma_{S,typ}$=1.35, although questionable, was taken as typical of all load effects on timber structures irrespective of their importance, and all members whether their mode of failure was in bending, compression, tension or shear and all laterally loaded nailed or screwed connections whether failure was due to fracture or slip. The comparisons also demonstrated that the assumption made in equation 5.21a, that k_{COM} = 1.0, was theoretically appropriate.

The subsequent satisfactory use of AS 1720.1-1997 appears to validate its calibration to AS 1720.1-1988 for the strength limit state via the conversion factor of 2.95.

5.4.8 Conclusions

The foregoing philosophical underpinning of the limit states design method for timber structures was largely constructed by CSIRO. It provides the most accurate means currently available for determining the theoretical probability of failure of timber structures. Four major developments affecting the reliability of timber structures resulted.

1) The concept of relative structure importance. The tool being the reliability index, β, that enables the relative theoretical probabilities of failure in a group of type structures to be compared with one another group, whatever their component materials.
2) A method of estimating the as-built strength of timber structures using limit state methods, that gives a probability of failure, (10^{-3}), sensibly similar to that obtained by working stress methods, for the reliability level of $\beta = 3.0$. This is obtained by applying the calibration factor of 2.95 to convert working stresses to characteristic stresses and applying the capacity factor, ϕ, to estimate structure resistance.
3) A method of characterizing structure components, i.e., scantling sized timber, reconstituted timber products and connections, by using a reliability factor, γ_M, that accounts for uncertainty due to inadequate sampling methods. The factor is applied to the lower 5%ile property values to give characteristic stresses, as determined by AS/NZS 4063-1992 *Timber- Stress-graded – In-grade strength and stiffness evaluation* and connector capacities, as determined by AS1649-1998, "*Timber - Methods of test for mechanical fasteners and connectors – Basic working loads and characteristic strengths.*"
4) The adoption of a single typical parameter to represent that part of the as-built reliability factor which accounts for variability in the various load effects

specified in AS 1170.1-1989. This parameter, $\gamma_{S,op}$=1.35, is applied to 5%ile characteristic stresses in AS/NZS 4063-1992 and AS 1649-1998.

The four developments are predicated on the transfer to AS 1170.1-1989 of the uncertainty in load effects as typically represented by $\gamma_{S,op}$. All other uncertainty causing unreliability in the structure is retained in either AS/NZS 4063-1992 or AS 1720.1-1997 and gathered into the resistance "material" and "structure" reliability factors, with the latter modified by β for levels of structure importance.

The above review of the calibration between working stress and design methods reveals that, as specified in AS 1720.1-1997, ϕ is a variable that simultaneously includes reliability factors, γ_I, to account for three theoretical strata of structure importance, and implicitly includes reliability factors, γ_B, for uncertainty in design and construction. However, as specified in AS/NZS 4063-1992, ϕ, when applied to material characterization, has a fixed value of 0.8 according to Appendix I, AS 1720.1-1997. These differing values of the capacity factor present a significant anomaly that requires explanation. In the remainder of this chapter, the writer examines in further detail the individual reliability functions that combine to form the capacity factor and a set of equations that removes the anomaly is obtained. Appropriate values are derived for the several reliability functions presently combined in the capacity factor.

5.5 A REVIEW OF THE DESIGN RESISTANCE EQUATIONS

5.5.1 Introduction

Without reference to AS 4063-1992 and the literature, an appreciation of the influence that the several components of the calibration factor exert on the design process is unavailable to an engineer designing a timber structure. Their influence needs clarification as a practical necessity, for it leads to an explanation of the apparent anomaly presented by the differing values of the capacity factor used to characterize material / components and estimate design resistance.

The absence of definitions of the several functions performed by the capacity factor obscures their individual effects on estimates of structure resistance and contributes to the anomaly. This section of the chapter assesses the composition and use of ϕ and proposes a more transparent set of equations that remove the anomaly. The equations enable an engineer to better judge where design and construction effort may be best directed to achieve the degree of reliability demanded by the structure under consideration.

5.5.2 Estimating Structure Resistance

The resistance of timber structures to the load effects specified in AS 1170.1-1989 is estimated by reference to three standard specifications; AS/NZS 4063-1992, AS1649-1992 and AS 1720.1-1997. The former two characterize material / component strength and elasticity, the latter prescribes the limiting stresses and elastic moduli for use in structural design. Their use is examined individually below.

- AS/NZS 4063-1992:

By substituting $\phi_{0,3.0} = 1.3 + 0.7V_R$, (equ.5.20a), and $\gamma_{S,op} = 1.35$, the limit states characteristic 5%ile stress expressed by equation 5.29b becomes

$$f_o' = \frac{\gamma_{S,op}}{\phi\gamma_{0.3.0}} f_{0.05,true} \qquad 5.31a$$

where $\gamma_{0,3.0}$ is the estimated structure reliability factor when $\beta = 3.0$.

- AS1649-1998:

The 5%ile load capacity of nails and screws are specified for limit states design as

$$Q_k = 2.95Q_{Basic} \qquad 5.31b$$

- AS1720.1-1997:

Substituting in equations 5.10, f_0' as given by equation 5.31a, the estimated 5%ile design resistance of

a timber component is $$R^* = \phi R_{0.05} = \frac{\gamma_{S,op}}{\gamma_{0,3.0}} \Pi k f_{0.05,true} X \geq S^* \qquad 5.32a$$

and a joint is $$R^* = \phi N_j = \frac{\gamma_{S,op}}{\gamma_{0,3.0}} \Pi knQ_k \geq S^* \qquad 5.32b$$

Equations 5.32 reveal that the capacity factor has been cancelled. This condition is clearly an unintended anomaly and is removed by recognizing that ϕ has two distinct, but quite unrelated values. One applied to resistance estimates in AS 1720.1-1997, the other to material characterization in AS/NZS 4063-1992. The value of ϕ for the former is that given by equations 5.19 or 5.20. The value of ϕ for the latter was subsequently amended to become the undefined numerical constant of 0.8 specified in AS 1720.1-1997, Appendix I. Substituting ϕ=0.8 in equation 5.32a gives

$$R^* = \frac{\gamma_{S.typ}}{0.8\gamma_{0,3.0}}\phi\Pi k f_{0.05,true} X \qquad 5.33a$$

and

$$R^* = \frac{\gamma_{S.typ}}{0.8\gamma_{0,3.0}}\phi\Pi k n Q_{k,j} \qquad 5.33b$$

At the calibration point, where β=3.0 and V_R=0.4, the ratio $\frac{\gamma_{S.typ}}{\gamma_{0,3.0}}=\frac{1.35}{1.58}$=0.85, which, when substituted in equations 5.33, gives

$$R^* = 1.06\phi\Pi k f'_{0.05,true} X \qquad 5.34a$$

and

$$R^* = 1.06\phi\Pi k n Q_k \qquad 5.34b$$

Equations 5.34 indicate that the calibration factor is 6% high. In itself, the implied inaccuracy is of no real concern for structural design generally, but it raises a question over its derivation. The matter is clarified in the following discussion.

5.5.3 The Calibration Factor.

5.5.3.1 General

The calibration factor adopted by the Australian code committee has a numerical value of 2.95. Individually, the function of each of the factors $\gamma_{S,typ}$, γ_R (=1/ϕ), and $\gamma_{0,3.0}$, that together comprise this value, is obscure and confused. The obscurity and confusion is primarily due to their want of definition. That is, a definition of

- γ_R ($=1/\phi$) that describes its dual function as a factor that; (a) controls the quality of design and construction, and (b) stratifies structure importance.
- $\gamma_{S,typ}$, as typical of the reliability of load effects factors.
- $\gamma_{0,3.0}$ as representing the overall in-service reliability of structures that possess a notional probability of failure of 10^{-3} in service.

No such definitions presently exist in either AS1720.1-1997 or AS/NZS 4063-1992.

As expressed by equations 6.30, the calibration factor consists of four components;

- the fixed duration of load factor, k_l=0.57,
- the fixed typical load effects parameter, $\gamma_{S,typ}$ = 1.35,
- the overall in-service structure reliability factor, $\gamma_{0,3.0}$ that has a value of 1.58 at the calibration point where β=3.0 and V_R = 0.4.
- the undefined numerical constant, ϕ = 0.8,

Each component is reviewed in the following discussion.

5.5.3.2 Duration of Load Factor, k_1=0.57.

The duration of load factor accounts for the reduction in strength that occurs when timber is under sustained load for 50 years. The in-service reliability factor, $\gamma_{0,3.0}$, is unaffected by k_l, as both are based on characteristic properties determined under the standard test conditions that require the test load to be applied during a 3-5 minute period at 20°C and 65%rh. The calibration assumes that k_l has a value of 0.57 of characteristic short-term ultimate strength for structures constructed from either sawn timber, glulam, laminated veneer lumber or plywood. The calibration makes no reference to particleboard.

5.5.3.3 Typical Load Effects Parameter, $\gamma_{S,typ}$

A strength limit state as typically expressed by equation 5.15a is

$$R^* = R_{0.05} / \gamma_R = \gamma_S S_{0.95} \geq S^*$$

When applied to resistance estimated by limit states methods, the load effects factor, γ_S, for a particular loading condition, effects a balance between limit states and working stress design outcomes. Hence, a typical value to represent the various load combinations is needed. A true typical value is not entirely feasible across the full range of possible load combinations and a compromise is used for calibration purposes.

The typical load effects parameter, $\gamma_{S,typ}$, is defined as the ratio

$$\gamma_{S,typ} = \text{limit states load effects / working stress design loads}$$

Any evaluation of $\gamma_{S,typ}$ is thus viewed as entirely deterministic, drawn from decades of engineering experience with the working stress design methods specified by AS 1720-1988 and its predecessors. Because structures so designed were generally satisfactory, and since for a limit state $R^* = S^*$, structures designed by limit states methods to AS 1720.1-1997 require that the ratio, $\dfrac{S^*_{LSD}}{\gamma_{S,typ} S^*_{WSD}}$ is unity, so that $$\gamma_{S,op} = \frac{S^*_{LSD}}{S^*_{WSD}} \qquad 5.35$$

Design loads, S^*_{WSD}, as specified by the working stress loading code, AS 1170_{WSD}, had no factors of safety applied to them overtly.

That is $$S^*_{WSD} = G + Q \qquad 5.36$$

where G is the total dead load.

Q the sum of various live loads, each containing their respective peak loads in a 50 year period.

For limit states design to AS 1720.1-1997, each component of the design load effect, S^*_{LSD}, has a partial factor applied to it as specified by the limits states version of the loading code, AS 1170.1-1989.

That is $$S^*_{LSD} = \gamma_S S_{0.95} = \gamma_G G + \gamma_l Q + \gamma_{sl} Q + \gamma_c Q + \gamma_W W_u + \gamma_{eq} F_{eq} \qquad 5.37$$

where $\gamma_G G$ effect of dead loads.

$\gamma_l Q$ effect of a long term live load.

$\gamma_{sl} Q$ effect of a short term live load.

$\gamma_c Q$ effect of combinations of live load.

$\gamma_W W_u$ effect of wind.

$\gamma_{eq} F_{eq}$ effect of earthquake.

Dead loads, although highly variable, are normally over estimated by the designer. Their conversion to factored dead load effects is presently effected by a single factor common to all dead loads, which implies a reasonable certainty that the estimated maximum dead load is rarely exceeded. However, time dependent live loads bring a much higher probability that underestimates of their effects would be made, to reduce which, they are subdivided into types and allocated individual factors. Variability in estimates of dead and live load effects are around 0.1 and 0.25 respectively, (Lam 2001), which gives a nominal combined load effects variability of 0.27. The probability that G and Q contain the peak load during the service life of a structure is accounted for in the values assigned to the ratio, γ, of the 95%ile of the peak load in one year to the estimated peak load during fifty years.

The idealization of applied loads to representative load effects and the neglect of non-assessable actions is also probabilistically embodied in γ. As approximate means of their ranges derived from available surveys, some of which are quite limited in scope, load factors are conservatively assessed before being adopted. The resulting factors specified in AS 1170-1989 are generally consistent with those under consideration by the International Standards Organisation for limits states design through the standard, ISO 2394:1998, *General principles on reliability for structures,* and will be refined as more data accumulates.

The most common load effect involving live loads is that specified in AS1170.1-1989 for the strength limit state as

$$S^{*}_{LSD} = 1.25G + 1.5Q \qquad 5.38a$$

A revision of this load effect that is currently under consideration proposes

$$S^*_{LSD} = 1.2G + 1.5Q \qquad 5.38b$$

Combining equations 5.35 and 5.36 with 5.38a and 5.38b respectively

$$\gamma_{S,op} = \frac{1.25G + 1.5Q}{G + Q} \qquad 5.39a$$

and $$\gamma_{S,op} = \frac{1.2G + 1.5Q}{G + Q} \qquad 5.39b$$

Values of $\gamma_{S,op}$ obtained from equations 5.39 are given in Table 5.11.

Table 5.11 Values of $\gamma_{S,op}$ related to the ratio of limit states load effects (specified in AS1170) to unfactored total working stress loads.

$G/(G+Q)$	$\gamma_{S,op}$	
	$(1.25G+1.5Q)/(G+Q)$	$(1.2G+1.5Q)/(G+Q)$
0.2	1.45	1.44
0.3	1.43	1.41
0.4	1.40	1.38

For timber structures the ratio of dead load to total load, $G/(G+Q)$, generally falls between 0.2 and 0.4, for which equations 5.39 gives a range of $\gamma_{S,op}$ from 1.38 to 1.45. A value of $\gamma_{S,op}$=1.42 obtains when the ratio is 0.33. ASTM D 5457-1998, *Standard specification for computing the reference resistance of wood-based materials and structural connections for load and resistance factor design*, assumed an appropriate ratio was $Q = 3G$, *i.e.*, $G/(G+Q)$= 0.25 which gives $\gamma_{S,op}$=1.44. For practical purposes the ASTM factor equals those obtained from equations 5.39. The ratios of $G/(G+Q)$

indicate that $\gamma_{S,op}$ should lie between 1.4 and 1.45. A single, conservative, value of 1.35 was adopted by Leicester (1980b) for calibration purposes, allowing room for modification as confidence in reliability theory improves.

Currently, a review of AS 1170.1, 1992 is giving consideration to raising the load factor for dead load to 1.40, which together with the foregoing data, indicate that the value of $\gamma_{S,op}$ should increase to 1.40. The value of the calibration factor is raised marginally from 2.95 to 3.0 as a consequence.

Even though γ_S is dedicated solely to uncertainty in load effects, the parameter $\gamma_{S,op}$ has been applied wholly to the characterization of material properties through AS/NZS 4063-1992 and AS 1649-1998. This conflicts with Steven's (1975) warning against the complications in design procedure that would arise *"if material and action effects load factors are coupled in anyway"*. It is evident that this is inappropriate if the essential separation between materials and load effects is to remain. Part of the lack of clarity regarding $\gamma_{S,op}$ is attributable to its forming part of the property characterization equation rather than the resistance equation.

5.5.3.4 Overall In-Service Structure Reliability Factor, $\gamma_{0,3.0}$.

Leicester concluded that secondary structures, including house framing, have a reliability index of $\beta = 3.0$ when made from timber with a variability of $V_R = 0.4$, "*in-place, in-service*". This condition defines the principal calibration point used by Leicester, at which, the reliability of structures designed by working stress methods theoretically matches that produced by limit states methods. Thus, putting structure variability V_R=0.4 in equation 5.20a, results in an overall structure reliability factor of $\gamma_{0,3.0}$=1.58 at the calibration point.

The materials reliability factor γ_M is removed from the resistance estimating methodology because its effect is included in material / component characterization, i.e., in determining $f_{0.05,true}$, γ_I is given unit value when β=3.0. Hence, at the calibration point, the general expression (equ.5.8c) may be written

$$\gamma_{0,3.0} = \gamma_{R,3.0}\gamma_{S,typ} \qquad 5.40$$

Substituting $\gamma_{0,3.0}$=1.58 and $\gamma_{S,typ}$=1.35 in equation 5.40 leads to $\gamma_{R,3.0}$=1.17. Therefore, the capacity factor $\phi_{R,3.0}$, which is the reciprocal of $\gamma_{R,3.0}$, has a value of 0.85. Thus, at the calibration point, the estimated resistance of a member or structure is given by

$$R^* = \phi R_{0.05} = 0.85\Pi k\left[f_{0.05,true}X\right] \qquad 5.41a$$

and of a joint by

$$R^* = \phi N_j = 0.85\Pi k Q_{k,j} \qquad 5.41b$$

It is noted that equation 5.26b gives the value, $\phi_{R,3.0}$=0.83. For practical purposes, either value of $\phi_{R,3.0}$, 0.83 or 0.85, holds the calibration factor to 2.95.

The foregoing division of the overall reliability of a structure, ϕ_0, indicates that, at the calibration point, around 55% is governed by uncertainties in load effects, $\phi_{S,typ}$, and 45% by uncertainties in estimating structure resistance, ϕ_R. Thus, the effect of codifying uncertainty in load factors is to remove nominally half of the design engineer's control over a structure's in-service reliability.

5.5.3.5 The Undefined Numerical Constant, $\phi = 0.8$

Although specified by Appendix I of AS 1720.1-1997 to have a value of 0.8, no reason is given in the literature for applying ϕ to material characterization (equation 5.28a). This stands in contrast to the use of ϕ as a variable factor applied to estimates of structure resistance in equations 5.10. If the undefined constant is omitted, the calibration factor of 2.95 reduces to 2.36 making resistance estimates deficient by 25%, i.e., by one stress grade and results in a reliability index of β =2.5.

Pearson (CSIRO (1958), in equation 5.11b, applied a factor of safety of 0.8 to properties characterized as one-percentile values. Hence, to achieve the same level of reliability in properties characterized to five-percentile values, the factor of safety reduces in the ratio of the standard deviations of the 5%ile and 1%ile values. That is, in the ratio $1.96\sigma/2.58\sigma$, the value of which is 0.76 assuming a lognormal distribution. As a consequence, equation 5.14 becomes

$$\frac{\overline{R}}{\overline{S}} = 2.52\frac{S^*}{\overline{S}} \qquad 5.42$$

and substitution in equation 5.3b gives the reliability index as

$$\beta = \ln 2.52\left(\frac{S^*}{\overline{S}}\right)\Big/0.75(0.4 + V_S) \qquad 5.43$$

Taking the ratios of $\frac{S^*}{\bar{S}}$ given in Table 5.5 for the three load effects and putting V_S=0.4 the results in an average nominal value of β=2.5, which when substituted in equation 5.18b, gives a load effects factor, γ_S=1.22. Hence, when β=2.5, an overall structure reliability factor $\gamma_{0,2.5}$=1.27 is obtained from equation 5.19 when γ_S=1.22 and $V_R = 0.4$.

The ratio of the overall reliability factors for reliability indices of β= 2.5 and 3.0 when $V_R = 0.4$ is thus

$$\frac{\gamma_{0,2.5}}{\gamma_{0,3.0}} = \frac{1.27}{1.58} = 0.8 \qquad 5.44$$

which is the value of ϕ needed to yield a calibration factor of 2.95 in equation 5.30a. That is, the capacity factor, $\phi_{2.5}$, which is the inverse of the resistance reliability factor, $\gamma_{R,2.5}$, has a value of 0.8.

In terms of the foregoing explanation, failing to reduce Pearson's safety factor, based on one-percentile characterizations, to compensate for the lower reliability of five-5%ile characterizations, the calibration procedure leads to a reliability index which is 25% lower than that needed to meet the calibration point for housing structures. To raise the housing structure index to β=3.0 and simultaneously retain the calibration factor of 2.95, it is necessary to insert the constant, ϕ=0.8, into the denominator of equation 6.30a. Hence, as a means of satisfying the calibration, ϕ=0.8 is used by AS/NZS 4063-1992 to raise 5%ile characteristic property values by 25%. The efficacy of raising property levels by these means is discussed in para. 5.5.6.

5.5.4 Removing the Anomaly

The capacity factor ϕ performs three independent functions. As currently used in AS1720.1-1997, ϕ grades estimates of structure resistance in terms of;

1. its importance based reliability, and
2. the variability of its component materials.

And, in AS/NZS 4063-1992, the undefined numerical constant ϕ is the means of

3. raising the reliability index for secondary structures and house framing from β=2.5 to β=3.0.

Substituting ϕ=0.85 (para 5.5.2) as the value of the undefined constant in the material characterization equation, 5.29a, gives

$$f_o' = \left(\frac{\gamma_{S,typ}}{0.85\gamma_{0.3.0}}\right) f_{0.05,true} \qquad 5.45$$

The expression, $\frac{\gamma_{S,typ}}{0.85\gamma_{0,3.0}}$, embraces the modifying factors applied to material characterization in AS/NZS 4063-1992.

Values of this expression, obtained by substituting material/component variability, V_M, in equation 5.20a, are given in Table 5.12.

Table 5.12 Values of the Modification Factor applied to Material Characterization.

V_M	$\gamma_{0,3.0}$	$\dfrac{\gamma_{S,typ}}{0.85\gamma_{0,3.0}}$
0.1	1.37	1.16
0.2	1.44	1.10
0.3	1.51	1.05
0.4	1.58	1.00

By combining and rearranging equations 5.21a and 5.45, the 5%ile characteristic resistance of a structure with a reliability index of β=3.0 becomes

$$R^* = \phi\left(\frac{\gamma_{S,typ}}{0.85\gamma_{0,3.0}}\right)\Pi k f_{0.05,true} X \qquad 5.46$$

and when V_M=0.4, equation 5.46 reduces to

$$f_0' = f_{0.5,true} \qquad 5.47$$

That is, at the calibration point, (β=3.0, V_M=0.4), the value of $\dfrac{\gamma_{S,typ}}{0.85\gamma_{0,3.0}}$ is unity for materials/components characterized to the 5%ile value. Hence, substituting f_0' for $f_{0.5,true}$ gives

$$R^* = \phi \Pi k f_0' X \qquad 5.48$$

which is the design resistance equation specified in AS 1720.1-1997.

5.5.5 Alternative Assessment of Calibration Factor.

For working stress design, the 5%ile basic stresses are given by equation 5.26b. Combining equation 5.26b with 5.20a and rearranging

$$f_{0.05,true} = 1.75\gamma_{0,3.0} f_{BASIC} \qquad 5.49$$

For limit states design, the 5%ile characteristic stresses are given by equation 5.29b. Substituting equation 5.20a in equation 5.27b and rearranging gives

$$f_{0.05,true} = \phi\gamma_{0,3.0} f_0'/1.35 \qquad 5.50$$

By definition $\phi_{R,3.0} = 1/\gamma_{R,3.0}$

and $\gamma_{0,3.0} = \gamma_{S,typ}\gamma_{R,3.0}$

Hence, by substitution $f_{0.05,true} = \gamma_{S,typ} f_0'/1.35$ 5.51

Substituting the value of 1.35 for $\gamma_{S,typ}$

gives $f_{0.05,true} = f_0'$

and the calibration equation becomes

$$f_0' = 1.75\gamma_{0,3.0} f_{BASIC} \qquad 5.52$$

Hence, at the calibration point, (β=3.0, $V_R = 0.4$), where the overall structure reliability factor $\gamma_{0,3.0}$=1.58, the calibration is given by

$$f_0' = 2.77 f_{BASIC} \qquad 5.53$$

This indicates that house framing and secondary structures are nominally 6% under-designed using the characteristic stresses specified in AS 1720.1-1997. The value, 1.06 × estimated design strength, agrees with the conclusion drawn from equations 5.34. In view of the difficulty of assessing the mitigating effects of redundancy, this result is acceptable for house frames and secondary structures.

However, it is not entirely acceptable for primary structures or primary members. This is because the design of most timber structures is governed by deformation limits which are reached before strength limits, particularly in redundant structures. Framed structures generally and pre-cambered beams fall into the category where deflection does not control and load sharing between adjacent members is unlikely. Therefore, if valid, the calibration should also suit the condition represented by structures with a reliability index of 3.5 and a variability of 0.2 in-service.

To test validity, consider primary members such as trusses, for which β=3.5 and V_R=0.2, (Table 5.8). The structure reliability factor is $\gamma_{0,3.5}$=1.68, (equation 5.20b) and the calibration becomes

$$f_0' = 2.94 f_{BASIC} \qquad 5.54$$

For practical purposes, this fully supports the calibration factor developed by Leicester for secondary structures, and confirms its suitability for primary structures or primary members / components too.

5.5.6 Discussion

Leicester (1980) stated that, *"the aim of in-grade testing is to evaluate the reliability of graded timber in-place in-service."* To achieve this aim, sampling methods, test methods and statistical techniques were devised to characterize sawn timber scantling. Test procedures allow for the probability that defects, as stress raisers, could be located to create the theoretically largest stresses in service. The effect of sample size is accounted for by the reliability factor γ_M, (equation 5.25).

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Leicester's methods for characterizing material / component strength, properly, exclude the effects of human error in design and construction activity. But, in the absence of any provision for these effects, the efficacy of the estimates of structure resistance may be questioned. As indicated by Table 5.14, the strength of material / components with low variability may be significantly over-estimated. Yet, no reason is given in the literature for raising strength above the 5%ile value. This is unacceptable, because the theoretical probability of failure is reduced as a consequence and without explanation. Further, in a public review of AS/NZS 4063-1992, it is proposed that ϕ be removed from equation 5.28a and replaced by the undefined numerical constant 0.8, that is, by the value of $\gamma_{R,3.5}$ to give characteristic stresses of

$$f_o' = 1.69 f_{0.05,true} / \gamma_{0,3.0} \qquad 5.55$$

This proposal, in failing to explain the reason for applying $\frac{\gamma_{S,typ}}{0.8\gamma_{0,3.0}}$ to material characterization, adds to, and perpetuates, the lack of clarity in design procedure. The issue is one which militates against error-free design practice.

5.5.7 Conclusions

It is concluded that;

- The resistance equation 5.48 is true only at the calibration point where β=3.0, V_M=0.4, and when the material modifying function $\frac{\gamma_{S,typ}}{0.85\gamma_{0,3.0}}$=1.0. That is, when $\gamma_{S,typ}$=1.35, $\gamma_{0,3.0}$=1.58 and ϕ=0.85.

- The anomaly between AS 1720.1-1997 and AS/NZS 4063-1992 is removed by eliminating the material modifying function $\frac{\gamma_{S,typ}}{0.8\gamma_{0,3.0}}$ from material / component characterization.
- At the calibration point, the value of γ_M is unity for characterization to the lower 5%ile probability limit, provided the sample size is truly representative.
- The capacity factor needs re-evaluation to account for the effects of human error in design and construction procedures.

In view of these conclusions, the capacity factor is re-evaluated in the following section.

5.6 RE-EVALUATION OF THE CAPACITY FACTOR ϕ.

5.6.1 General

Equations 5.23 indicate that the capacity factor varies according to a structure's importance, member strength properties and variability due to human error in design and construction.

The concept of structure importance was introduced into structural timber design methods to enable a differentiation to be made between the reliability of various structures by assigning, through the reliability index β, levels of importance appropriate to the risk of failure and its effect on community welfare. The concept also enables appraisals of selected critical and non-critical members and connections to be made individually. No reliability factors for structure importance, γ_I, are explicitly specified in AS 1720.1-1997, they are implicit in the values of ϕ, (Table 5.10 this text), that are varied in terms of β.

Although the matter is fundamental to sound design practice, no explicit provision for human activity that produces design and construction error exists in AS 1720.1-1997. The reliability factor, γ_R, provides for human error, but, as currently applied in AS 1720.1-1997 via its reciprocal ϕ, it is not possible to place a value on any effort expended to reduce design and construction variability beyond that which is generally spent on house framing.

From a designer's viewpoint, sources of error in design and construction and member or the relative importance of a structure are independent issues that should be treated separately. To achieve this separation, a re-evaluation of ϕ is undertaken in the following discussion.

5.6.2 Dual Function of the Capacity Factor ϕ.

Grading structures for the quality of design and construction procedures and for structure importance is effected by dividing ϕ into two independent variables, Φ for the former, 1 for the latter. The role of the reliability index β is thus clarified by inserting it in the form of 1 in the resistance equations.

Substituting the product, ϕ_B I, for ϕ in equation 5.48, the estimated 5%ile characteristic resistance of a structure, or a member / component, with a reliability index of β=3.0 is

$$R^* = \phi_B \mathrm{I}\Pi k f_0' X \qquad 5.56a$$

and for a joint $$R^* = \phi_B \mathrm{I}\Pi k n Q_{k,j} \qquad 5.56b$$

5.6.3 Design and Construction Activities Contributing to Structural Failure.

The majority of structural failures involve human error in design and construction procedures, the reduction of which may be assisted by intervention through quality assurance programs. Such management systems aim to improve the reliability of the activities involved by formalizing work practice and setting regimes for

1) design briefing and supervision.
2) documentation checking and contract preparation.
3) construction supervision.

As the means of reducing human error, the foregoing reduce simply to the use of higher levels of practice skill and experience, which, as the following discussion illustrates, provide the best means of increasing reliability by reducing mistakes.

Pugsley (1973) concluded that "*proneness to structural accidents arising from human error*" could be attributed to activities placed in the following order.

1) New or unusual materials.
2) New or unusual methods of construction.
3) New or unusual types of structure.

4) Experience and organization of design and construction teams.

5) Research and development background.

6) Financial climate.

7) Industrial climate.

8) Political climate.

The first five activities affect the design process directly and are ostensibly under the control of the engineer responsible for design – ostensibly because on occasion, field decisions are taken that negate the design intent. Items 2, 4 and 7 affect the construction process which frequently may not be so controlled, e.g., stopping concrete supply partway through a "pour". Item 6 affects both design and construction by the contractor's financial viability determining the extent and quality of the work possible, while the last two, although beyond the control of the design engineer, may have unpredictable effects militating against safety, e.g., due to strike action or client constraints on engineering design and supervision.

Pham and Leicester (1979) referred to Allen (1977) who reported that a survey of structural failures revealed that about half those known were due to design errors, the other half to construction errors. Walker (1981) also identified the primary causes of structural failure as products of human error and weighted them according to Table 5.13. Implicit in these subdivisions are the adverse effects attributable to poorly characterized material properties that affect both design and construction.

Table 5.13 Prime "causes" of failure, (after Walker 1981).

Item	Cause	%
1	Inadequate appreciation of loading conditions or structural behaviour	43
2	Inadequate execution of erection procedure.	13
3	Random variations in loading, structure, materials, workmanship, etc.	10
4	Contravention of requirements in contract documents or instructions.	9
5	Mistakes in drawings or calculations.	7
6	Inadequate information in contract documents or instructions.	4
7	Unforeseeable, misuse, abuse, sabotage, catastrophe, deterioration.	7
8	Others.	7

Melchers (1987) developed a complementary classification to Walker's with the proportionate effects of human error indicated in Table 5.14.

Table 5.14 Human error factors in observed failure cases, (after Melchers 1987).

Item	Factor	%
1	Ignorance, carelessness, negligence	35
2	Insufficient knowledge	25
3	Underestimation of influences	13
4	Errors, mistakes, forgetfulness	9
5	Reliance on others without control	6
6	Unimaginable situations	4
7	Other	8

Melchers' classification of the causes of failure was independent of whether the activity was associated with either design or construction and a nominally equal subdivision may be inferred between ignorance, carelessness, mistakes and forgetfulness (Items 1 and 4 total 44%), and misjudgements (Items 2, 3 and 5 total 43%). Whilst not so clear-cut, Walker's classification also attributes failures as shared between design error and construction error in roughly equal proportions. By taking Items 1, 3, 5, 6, as designer controlled, a nominal 2:1 ratio results which reduces to around 1:1 when a reasonable proportion of Items 1 and 3 is attributed to construction inadequacies.

It is concluded from the foregoing that the reliability of a structure in service is

(a) nominally doubled by the use of procedures that overcome mistakes, thereby halving the variability in design and construction, and

(b) apportioned equally between human error in the design office and in the field.

While these interpretations may be questioned, because without construction supervision, design procedures tend to be more conservative, the data tabulated reinforce the view, that increased practice skill, experience and supervision, whether applied to design or construction, reduces the probability of failure. Cautious engineers may be willing, therefore, to adopt modifying factors drawn from experience, that value rigorous design and more detailed construction supervision. Such factors are developed in the following discussion.

5.6.4 Human Error in the Design and Construction Process

5.6.4.1 General

The reliability of structures can only be assessed through design and building practice that has a verifiable history. Data specific to the variability in strength of timber structures due to human error in design and construction activity is sparse making it necessary to draw on the variability experienced with steel and concrete structures. The capacity factors developed in Section 5.6.5 to account for various grades of design and construction quality of timber structures largely reflect the central position that the design and fabrication of joints occupy in structure reliability.

The orthotropic nature of timber and its effect on the location of connectors, its response to moisture and its uncertain long term behaviour combine to make the design of timber connections an exacting, albeit, uncertain task. The size of a joint, as dictated by the spacing of connectors and their end and edge distances, will, for large connectors, usually require the installation of a member larger than that dictated by stress so that limiting stresses in a member are rarely reached before those in the joint are exceeded. Hence, uncertainty in structure resistance is primarily a product of uncertain joint behaviour as distinct from member behaviour. However, as discussed in Chapter Seven, connections made small with fasteners no larger than 4*mm* diameter are usually contained within a stress-sized member and will exhibit reasonably predictable ductile behaviour with low variability.

A significant contribution towards a better understanding of the effects of design and construction was made by the American Society of Civil Engineers (1948) in their report on tests of several large timber structures built in 1939 for the Golden Gate International Exposition. Each was designed by a different consulting engineer in the course of his normal practice and in the knowledge that it would be tested to destruction after the exposition. The program was interrupted in 1942 by the advent of World War 2, which saw the remaining untested buildings dismantled, stored under cover until they were reassembled at the University of California when testing recommenced in 1945. The work, completed in 1946, revealed that:

- Ruptured joints precipitated all structure failures.
- Predicted as-built factors of safety were not realized.
- As-built factors of safety were highly uncertain, ranging from 1.5 to more than 7.

Joint ruptures were observed to result from both design and construction deficiencies. In design, by inadequate provision for

1. tensile stresses perpendicular to the grain induced by moments caused by non-coincidence of member gravity axes and eccentric connector arrangements, and restraint offered by connectors to member rotation as the structure deforms, and
2. torsional stresses induced when members are lapped.

And in construction, by deficiencies due to

1. inaccurate fabrication ,
2. use of blunt tools,
3. installation of smaller or fewer connectors than those specified,
4. omission of connectors altogether,

5. loose connectors.

Alluding to member design, as distinct from joint design, Gould, a consulting engineer and prime author of the ASCE report, stated that the *"wholly satisfactory performance of exposition structures designed with 50 to 100% increases of the usual timber unit* (working) *stresses, suggests that where competent engineers make the design and supervise construction and maintenance, building codes should be revised to provide for increased unit* (working) *stresses - perhaps 30-50% more than now permitted."*

Referring to joint design, Smith and Foliente (2002) expressed the opinion that, without broader statistical data, the rigorous application of reliability theory to assess probability of failure for timber joints is not presently feasible. Uncertain joint behaviour can only be reduced through competent, detailed design and sound engineering judgment supported by adequate testing and adequate supervision during construction.

5.6.4.2 Variability in Design, V_D and Construction, V_C.

A reasonable approximation of total variability when variables are independent and normally distributed, may be expressed as

$$V_0^2 = V_1^2 + V_2^2 + \cdots$$

hence, variability in the resistance factors due to variability in design and construction may be considered as

$$V_B = \sqrt{V_D^2 + V_C^2} \qquad 5.57$$

where V_D is the variability in the quality of the design process.

V_C variability in the quality of the construction process.

V_B variability in the in the quality of the as-built structure.

The literature indicates that variability due to human error in each of design and construction, ranges between 0.1 and 0.4. The resulting range of variability in as-built resistance factors, V_B, computed from equation 5.57, is given in Table 5.15.

Table 5.15 Variability in Design, V_D, and Construction, V_C, related to As-built Variability, V_B.

V_C	V_D	V_B
0.10	0.10	0.14
	0.40	0.41
0.40	0.10	0.41
	0.40	0.57

To establish their affect on V_B, variability in V_D and V_C is considered individually in the following discussion.

5.6.4.3 Definitions.

To examine the variability in design and construction quality, it is necessary to define four quality levels, or grades, of engineering design and construction activity. The following definitions are limited to that part of the design process that remains within the of control the designer, because the mathematical processes involved in the transformation of loads into design actions are excluded from the designer's domain.

The latter are embodied in the value of $\gamma_{S,op}$ that is derived from the codified load factors specified by AS 1170-1992. While the following definitions apply specifically to timber structures, they could be readily redrawn to apply to structures generally. Grade D is the standard the writer believes to be the lowest acceptable for structural work. It is not possible, nor is it desirable from a practice standpoint, to similarly define the effects of sub-standard design, V_D>0.4, or unsupervised construction, V_C>0.4, which together or singly could raise V_B well above 0.5 and border on incompetence, possibly negligence.

The proposed four grades of quality in design and construction procedures are defined as follows;

- Grade A, $V_B = 0.1$, the highest standard of design and construction activity, represents the lowest variability achievable in as-built resistance, and is obtained through rigorous design supported by prototype testing and adequate construction supervision.
- Grade B, $V_B = 0.2$, obtains with rigorous design, but no testing, supported by adequate construction supervision. It has an as-built variability twice that of Grade A.
- Grade C, $V_B = 0.3$, obtains when a normal standard of design is supported by adequate construction supervision.
- Grade D, $V_B = 0.4$, has the same design variability as Grade C. But when acting with nominal construction supervision, which has twice the variability of Grade C supervision, as-built variability is raised some 30% above that of Grade C.

The following terminology is employed in the foregoing classification of design and construction activity.

Rigorous Design is the method of design that fully accounts, by computation or prototype test, for the secondary stresses induced in members and joints by eccentricity due to non-concentric arrangements of fixing units and members' gravity axes, and structure deformation - short and long term – which may set up large tensile forces perpendicular to the grain in joints with limited ductility and large bending stresses in members generally. It also accounts rigorously for possible variations in member rigidity that affects the distribution of bending, shear and direct stresses. Where structures are complex or otherwise intractable to accurate computation, prototype testing enables the failure mode to be identified and the design method modified to avoid or largely mitigate uncertain behaviour.

Normal Design is the standard of design employed normally by a competent engineer where provisions for secondary stresses and the effects of rigidity are not made rigorously and no testing is undertaken.

Adequate Construction Supervision is the level of oversight necessary to ensure that the design intent is fully satisfied in the completed structure.

Nominal Construction Supervision is the level of oversight necessary to ensure only that principal components are generally correctly sized and placed, but is unable to reliably ensure that the design intent in detail, particularly with respect to connections, is fully satisfied in the completed structure.

5.6.4.4 Variability in Computational/Mathematical Modelling, V_D.

Wood (1958) quantified mean values and standard deviations of the components of the factor of safety of as-built timber structures designed by working stress methods. The coefficients of variation of the various constituents of the design and construction process, as computed from his data, are given in Table 6.15. Dividing equally between design and construction variability the variability ascribed to load effects and "Other", the remainder of the safety factor is also seen to be equally divided.

Table 5.16 Coefficient of Variation of Factors of Safety used in the Design, Fabrication and Construction of Timber Structures. (after Wood 1958).

Factor	Mean value	Std. deviation	C.o.V
Load effects	1.10	0.08	0.26
Design	1.00	0.03	0.17
Fabrication	0.95	0.02	0.15
Construction	1.00	0.01	0.10
Other	1.00	0.07	0.26

Fabrication and construction variability combine to have a value of 0.18, practically equalling the design variability of 0.17. This reinforces conclusion (b) drawn from Walker (1981) and Melchers (1987) that variability is practically divided equally between design and construction. Wood's 1958 value of 0.26 for variability in load effects corresponds to the current 0.27 value, (V_S=0.10 dead load, 0.25 live load), Pham (2001).

Pham and Leicester (1979) subdivided sources of uncertainty within the design process into the five categories given in Table 5.17. Drawing on supplementary investigations by others, they assessed variability within each category by calibrating the designs for a limited number of commonly used joints and structures against full scale tests to destruction.

Table 5.17 Coefficients of Variation, V_D, in Design Uncertainty. (after Pham and Leicester 1979)

Source of design uncertainty in a group of structures	V_D
Design, one designer, one theory, one structure type	0.10-0.30
Theory, different designers, different theories	0.10-0.30
Group, different structure types, one theory	0.15-0.35
Designer, different designers	0.20-0.50
Discrete, material sizes used v those computed	0.05-0.15

Total variability was found to range from 0.15 to 0.50, usually with only one or two of the five categories of uncertainty predominating for a particular structure. Short of gross negligence, it appears that simple structures normally designed by standard methods exhibit V_D around 0.3 rising to 0.5 occasionally. The Discrete item is a design rather than a construction deficiency should the designer fail to recognize the not uncommon use of scant-cut timber in a structure. Apart from this aspect, material size has no detrimental influence on reliability because it is standard engineering practice to substitute a larger size for a smaller, but unobtainable, specified size. While Pham and Leicester (1979) made no direct assessment of the quality of design, a number of important influences implicitly affect design variability; the time available and the care taken, the number of alternative structures considered, the engineer's experience with the type of structure, the amount of checking undertaken, the amount of testing undertaken, the extent of design detail prepared for fabrication - connections in particular - and transcription of the design into documentation suitable for construction. All reflect the degree of professional practice skill allotted to the task and all affect design reliability and the cost of the design documentation prepared for construction purposes.

The reliability of the construction process, i.e., shop fabrication and field erection, also affects as-built strength, (Pugsley 1973, Walker 1981, Melchers 1987), and because both were often beyond the designer's control, the designer commonly offset any anticipated reduction in as-built strength by specifying over-strength or over-size components in an attempt to retain an equivalent level of reliability. As a corollary, there is ample anecdotal evidence that inadequate construction supervision militates against any extra design effort that may be expended to raise the level of reliability while simultaneously endeavouring to hold down construction costs. Reference to the following Tables 5.19 and 5.25 confirm this view.

Taking the overall factor of safety as equally divided between design and construction, an adequate regime of construction supervision could reduce as-built variability by around 25%. Alternatively, the use of more rigorous design methods could also reduce as-built variability a similar amount.

5.6.4.5 Variability in Fabrication and Construction, V_C.

Pham and Leicester (1979) listed a number of typical coefficients of variation that accounted for variability in complete structures or their elements as given in Table 5.18. These were supplemented by commonly accepted values drawn from other sources.

Table 5.18 Coefficients of Variation V_B of Fabricated Structures / Components.

Fabricated structure or component	Coefficient of variation V_B
Steel box girders	0.07
Reinforced concrete joints	0.07
Timber aircraft wings and fuselages	0.07
Fillet welded and bolted steel joints and members	0.10
Steel transmission towers	0.10
Reinforced concrete beams and floor slabs	0.15
Nailed and nail-plated timber joints	0.15
Timber trusses, timber columns and finger joints	0.20
Bolted timber joints	0.30

The low values of 0.07 for aircraft structures, box girders and concrete joints and 0.10 for steel transmission towers and field jointing, are attributable to design theories being fitted to prototype testing specific to those structures and joints. Evidently, the design uncertainty inherent in complex structures that are not amenable to normal computational techniques, is nominally halved when rigorous design is supported by prototype testing. But, as indicated in Table 6.18, there is no evidence that prototype testing measurably reduces uncertainty when normal simple structures are rigorously designed, ($V_D = 0.10$). It is concluded, that provided both are adequately supervised, structures rigorously designed will exhibit nominally half the variability of those normally designed.

5.6.5 Capacity Factors, ϕ_B, Structure Importance Factor, I, and Overall Structure Reliability Factor, γ_0.

5.6.5.1 Design and Construction Factor, ϕ_B.

To establish capacity factors for design and construction, it is necessary to establish appropriate limits to variability in these activities. The capacity factors specified in Table 2.5, AS 1720.1-1997 range between 0.90 and 0.60. They are founded on the design and construction of house framing, but, as the following discussion reveals, could be too high to "save" normal design and construction procedures when used for primary and essential services structures. Provided unimaginable events, gross errors and negligence are excluded, reasonable limits for variability in as-built design and construction quality may be established by using the coefficients of variation in design uncertainty given in Table 5.17, (Pham and Leicester 1979). Values of V_B corresponding to design and construction grades A-D were computed using equation 5.57 and are given in Table 5.19.

Table 5.19 Design and Construction Grades A-D and As-built Variability, V_B.

Design and construction quality	Grade	V_D	V_C	V_B
Rigorous design, prototype testing, adequate construction supervision.	A	0.10	0.05	0.11 **0.1**
Rigorous design, no prototype testing, adequate construction supervision.	B	0.20	0.10	0.22 **0.2**
Normal design, no prototype testing, adequate construction supervision.	C	0.30	0.15	0.33 **0.3**
Normal design, no prototype testing, nominal construction supervision (occasional inspection).	D	0.30	0.30	0.42 **0.4**

Adopting rounded (bold) values of as-built variability, V_B = 0.1, 0.2, 0.3 and 0.4 for design and construction Grades A, B, C and D respectively, and assuming that structure in-service strength is lognormally distributed, values of ϕ_B are computed from equation 5.23a by putting $V_B = V_R$. Hence, when $\beta = 3.0$,

$$\phi_B = \left(\overline{R} / R_{0.05}\right)e^{-2.25V_B} \qquad 5.58$$

Values of ϕ_B thus computed are given in Table 5.20.

Table 5.20 Design and Construction Factor ϕ_B related to As-built Variability V_B and Design and Construction Grades A-D for Reliability Index $\beta = 3.0$, (equ 5.58).

Design and Construction Grade	V_B	ϕ_B
A	0.10	0.94
B	0.20	0.90
C	0.30	0.86
D	0.40	0.83

These values of ϕ_B are reasonably close to those of ϕ specified for secondary structures in Table 5.10, this text, and in Table 2.5 and Table I1, Appendix I, AS 1720.1-1997.

As earlier noted, ϕ is regarded as a function of material/component variability in AS/NZS 4063-1992 and is also tied to structure importance by AS 1720.1-1997, whereas it is, correctly, a determinant of the quality of design and construction alone. The fusion of these functions into a single factor contributes significantly to the lack of clarity surrounding the use of ϕ.

5.6.5.2 Structure Importance Factor, I.

Values of the importance factor I for each level of structure reliability may be obtained by substituting V_B for V_R in equations 5.23. That is when

$\beta = 3.0$ $$\mathrm{I}_{3.0} = \left(\overline{R} / R_{0.05}\right)e^{-2.25V_O} \quad 5.59a$$

$\beta = 3.5$ $$\mathrm{I}_{3.5} = \left(\overline{R} / R_{0.05}\right)e^{-2.625V_O} \quad 5.59b$$

$\beta = 4.0$ $$\mathrm{I}_{4.0} = \left(\overline{R} / R_{0.05}\right)e^{-3V_O} \quad 5.59c$$

Thus, the incremental shift in structure importance, ΔI, between the three values of β is given by

$$\Delta I = e^{0.375V_O} \qquad 5.60$$

Taking the base reference value of I as unity when $\beta = 3.0$, values of I for $\beta = 3.5$ and $\beta = 4.0$ are proportioned to this base by equation 5.60. Resulting values of I are given in Table 5.21.

Table 5.21 Values of I Related to β and V_B.

V_B	$\beta = 3.0$	$\beta = 3.5$	$\beta = 4.0$
	$I_{3.0}$	$I_{3.5}$	$I_{4.0}$
0.1	1.00*	0.96*	0.92*
0.2	1.00	0.93	0.86
0.3	1.00	0.89	0.80
0.4	1.00	0.86	0.74

* Values of I appropriate for structural particleboard.

5.6.5.3 Theoretical Reliability of As-built Structures, γ_0.

The theoretical reliability factor γ_0 (equation 5.8b) for an as-built structure with a reliability index β=3.0, i.e., when γ_1=1.0, reduces to

$$\gamma_0 = \gamma_S \gamma_R \gamma_M \qquad 5.61$$

Substituting $1/\phi$ for γ_R, equation 5.61 may be written as

$$\gamma_0 = \frac{\gamma_{S,typ}\gamma_M}{\phi_B \mathrm{I}} \qquad 5.62$$

Variability in load effects is represented by $\gamma_{S,typ}$=1.35 and in material / components by the function $\gamma_M = 1 - \frac{2.7V_R}{\sqrt{n}}$. Hence, by putting γ_M=1.0 on the basis that the number of samples, n, is adequate, equation 5.62 reduces to

$$\gamma_0 = \frac{1.35}{\phi_B \mathrm{I}} \qquad 5.63$$

Values of γ_0 computed from equation 5.63 are given in Table 5.22.

Table 5.22 As-built Reliability Factor γ_0 related to Design & Construction Grades A-D, when Load Parameter $\gamma_{S,typ}$=1.35 and Duration-of-Load Factor k_l=0.57.

Design and Construction Grade	$\gamma_{0,3.0}$	$\gamma_{0,3.5}$	$\gamma_{0,4.0}$
A	1.44	1.50	1.56
B	1.50	1.61	1.74
C	1.57	1.76	1.96
D	1.63	1.89	2.20

The values of γ_0 obtained using equation 5.63, Table 5.22, are acceptably similar to those using equation 5.19 as given in Table 5.7. Bearing in mind, that, as the theoretical probability of failure reduces, the real value of γ_0 is becoming less determinate, the design outcome is, in reality, unaffected.

It is noted, that the removal of the material modifying function $\frac{\gamma_{S,typ}}{0.8\gamma_{0,3.0}}$ from the material / component characterization equations, 5.31, and, the introduction instead, of Φ into the design resistance equations, 5.56, is justified. It is also noted, that the degree of accuracy implied by 3-figure values of γ_0 imbues their derivation with unwarranted precision.

5.7 PROTOTYPE AND PROOF TESTING.

5.7.1 General

To satisfy building regulations, a structure may be deemed structurally competent by either computation or testing with each ideally providing an equal estimate of the level of structural resistance obtaining at any given time during its fifty-year service life.

Prototype testing is often used to assess structural competence of a group of similar structures. The aim is to minimise the uncertainty that a population of structures would fail to meet a specified limit state as determined by computation while minimizing the cost of the structure(s) represented by the prototype. The increased knowledge of structural behaviour so obtained when transposed into appropriate mathematical models, diminishes uncertainty and enables less costly structures with the same degree of reliability to be obtained. In this study, prototype testing is used to assess the adequacy of the theory used to design the wall beam models. Proof testing is a special case of prototype testing, useful for measuring economically the suitability of a unique structure, usually under service conditions, when its structural adequacy is in question.

5.7.2 Prototype Testing

AS 1720.1-1997 specifies the critical prototype test load equivalent, Q_E, to the specified limit state design load as

$$Q_E = k_{28} Q^* k_2 k_{26} k_{27} / k_1 \qquad 5.64$$

where Q^* is the (specified) design load or design action effect.

k_1 duration of load factor related to a 3 min. test period.

k_2 =1.0 (or 0.8) for connector redundancy in housing construction.

k_{26}=1.0 when the duration of load effect is similar to its effect on a simple beam.

k_{27} compensates for test periods in excess of 15min. to reach full load.

k_{28} sampling factor.

Q^* includes allowances for uncertainty in load effects, and factors k_1, k_2, k_{26}, and k_{27} for duration of load, structure type and test methodology. By default, the sampling factor, k_{28} accounts for all remaining uncertainty in member resistance, that is, the effects of material variability, variability of design and construction procedures, and the importance of a structure.

Because the bending, direct, and shear stresses induced are directly proportional to the applied load, equation 5.61 may be written in terms of structure resistance as

$$R_{\min} = k_{28} R^{*} \qquad 5.65$$

Substituting for R^{*} from equations 5.10

$$R_{\min} = k_{28} \phi R_{0.05} \qquad 5.66$$

where $R_{\min}$ the theoretical minimum strength of n similar structures.

The reliability of the sample prototype(s) to represent the population is given by the test reliability factor, γ_T, so that

$$R_{\min} = \gamma_T R_{0.05} \qquad 5.67$$

Equating equations 5.66 and 5.67 gives

$$k_{28} \phi R_{0.05} = \gamma_T R_{0.05} \qquad 5.68$$

therefore $$k_{28} = \frac{\gamma_T}{\phi} \qquad 5.69$$

Leicester (1987) gives γ_T as $\gamma_T = \left(\dfrac{27}{n}\right)^{V_{prot}}$ 5.70

where V_{prot} is the coefficient of variation of the prototype structure.

Hence $$k_{28} = \frac{(27/n)^{V_{prot}}}{\phi} \qquad 5.71$$

Values of γ_T are given in Table 5.23 for prototype variability between 0.1 and 0.4.

Table 5.23 Test Reliability Factor, γ_T, for Prototype Testing. (equ. 5.70).

No. of units tested. n	γ_T			
	V_{prot} =0.10	V_{prot} =0.20	V_{prot} =0.30	V_{prot} =0.40
1	1.4	1.9	2.7	3.7
2	1.3	1.7	2.2	2.8
5	1.2	1.4	1.7	2.0
10	1.1	1.2	1.4	1.5
20	1.0	1.0	1.1	1.1

Values of k_{28} assuming $\phi = 0.83$ are given in Table 5.24a together with corresponding values specified by AS 1720.1-1997, in Table D3 shown in italics for comparison.

Table 5.24a Sampling Factor k_{28} for Design and Construction Grade D, ($\phi = 0.83$)

No. of units tested. n	k_{28} for members			
	V_{prot} =0.10	V_{prot} =0.20	V_{prot} =0.30	V_{prot} =0.40
1	1.7 *1.6*	2.3 *2.4*	3.2 *3.5*	4.5 *5.2*
2	1.6 *1.5*	2.0 *2.0*	2.6 *2.8*	3.4 *3.9*
5	1.4 *1.4*	1.7 *1.7*	2.0 *2.2*	2.4 *2.7*
10	1.3 *1.3*	1.5 *1.5*	1.6 *1.8*	1.8 *2.1*
20	1.2 *1.2*	1.3 *1.3*	1.4 *1.4*	1.4 *1.6*

V_{prot} is the coefficient of variation of the individual prototype structure or component.

Values from AS 1720.1-1997 are given in italics.

The values of k_{28} provide a prototype test load, Q_E, that is specifically suited to structures built from timber or reconstituted wood products which meet design and construction Grade D, (ϕ_B=0.83, Table 5.20), and possess a reliability index of β=3.0.

As given in italics in Table 5.24a above, the values specified for members by AS 1720.1-1997, (Table D3) indicate a similar reliability index of β=3.0 but a standard of design and construction somewhat lower than Grade D. And as specified for joints, the values of k_{28} given in Table 5.24b assume an arbitrary value for ϕ of approximately 0.6, which reflects the greater uncertainty surrounding joint behaviour as compared with that of members.

Table 5.24b Sampling Factor k_{28} for Joints, (Table D3, AS 1720.1-1997).

No. of units tested. n	k_{28} for joints			
	V_{prot}=0.10	V_{prot}=0.20	V_{prot}=0.30	V_{prot}=0.40
1	1.8	2.9	4.8	-
2	1.7	2.6	3.9	-
5	1.6	2.1	2.9	-
10	1.5	1.9	2.4	-
20	1.4	1.6	1.9	-

V_{prot} is the coefficient of variation of the individual prototype structure or component.

Using cost optimisation concepts, Leicester, (1984a, 1986b) demonstrated that the values of k_{28} in Table 5.24a are consistent with assumptions that the relative cost of an in-service failure in a member to the cost of rectification is about five fold for highly redundant or secondary structures, and some fifty fold for primary structures. In the absence of broad based data on the cost of rectification, the validity of these ratios is open to question. Particularly the latter, where, for example, the failure of a roof truss in an industrial building could entail consequential costs very much greater than the cost of rectification alone.

5.7.2.1 Structure Importance

Equation 5.71 makes no provision for structure importance and provides values of Q_E for structures designed to meet a reliability index of $\beta = 3.0$. The difference in the respective values of k_{28} given in Tables 5.24 reflects the absence of The means of adjusting k_{28} to account for structure importance is provided by substituting the product, $\phi_B \mathrm{I}$, for ϕ, in equation 5.71, so that

$$k_{28} = \frac{(27/n)^{1/rw}}{\phi_B \mathrm{I}} \qquad 5.72$$

The effect that each variable has on k_{28} and hence on Q_E is discussed below.

5.7.2.2 Effect of Structure Importance Factor, I, on k_{28} and Q_E

Variation of the probability of failure, p_F, to account for relative structure importance in the test structure is accomplished by substituting in equation 5.69, the values of I given Table 5.21. Thus, k_{28} increases as the consequence of failure becomes more serious. The need for the cost of testing to be assessed in terms of the cost of failure, a task virtually impossible to generalize, is avoided by the use of I .

5.7.2.3 Effect of Capacity Factor, ϕ_B, on k_{28} and Q_E

Uncertainty in joint design and overall structure fabrication is taken into account by ϕ_B which grades design and construction rigour, therefore, specific values of k_{28} as given for joints in Table 5.24b are no longer necessary. Consequently, they are omitted from Table 5.23. Values of ϕ_B are given Table 5.20.

5.7.2.4 Effect of Prototype Variability, V_{prot}, on k_{28} and Q_E

Obtaining an appropriate value for k_{28} involves the selection of a suitable value of V_{prot}. Because the test sample must be stronger than the average structure, assuming a high variability for a given prototype leads to higher test loads and overly conservative structures. Consequently only prototypes that have low coefficients of variation should be selected for test. The endeavour then, is to ensure that all other elements in the prototype are stronger than the particular element being examined. This necessitates individual assessment of k_{28} for all members and joints. Theoretically, weaknesses revealed by testing enable appropriate strengthening to be effected until all elements in the prototype fail simultaneously, - a utopian objective. As indicated by Pham and Leicester (1979), Leicester (1987), where the variability of theory is high due to uncertain theory or ignorance of true structural action, prototype testing will provide more reliable structures at least cost. For simple structures, computational design is capable of giving least-cost structures because material and design variability will be better understood and thus better quantified.

5.7.2.5 Effect of Number of Prototypes Tested, n, on k_{28} and Q_E

CSIRO (1974) developed the values in Table 5.25 for the variability, V_B, in a prototype structure's as-built strength resulting from variability in the accuracy of the design theory, V_D , and variability in the strength of its assembled members and joints, V_C. Table 5.25 is similar to Table 5.19, which also relates design and construction quality to as-built variability.

Table 5.25 Coefficients of Variation for assessing the strength of Timber Structures, (after CSIRO 1974).

V_C	V_D	V_B for n prototype structures			
		$n = 0$	$n = 1$	$n = 5$	$n = \infty$
0.10	0.10	0.14	0.11	0.11	0.10
	0.40	0.41	0.14	0.11	0.10
0.40	0.10	0.41	0.41	0.41	0.40
	0.40	0.57	0.49	0.43	0.40

Several conclusions may be drawn from Tables 5.25 and 5.19. A 20% gain in reliability is obtained when rigorous design and adequate construction supervision are supported by as few as one prototype test. On the other hand, normal design with adequate construction supervision could require five tests for a 20% gain. Considering a structure rigorously designed but constructed under nominal supervision, it is evident that testing will add practically nothing to its as-built reliability. However, irrespective of the quality of material or the standard of design, there appears to be little practical gain in testing more than five prototypes without modifications that improve performance. For design by computation alone, V_D and V_C may be related to variability in as-built strength when no testing is undertaken by putting n = 0.

5.7.3 Proof Testing

To prove that a unique structure can withstand its unfactored design load, or some other selected action effect, it is subject to a proof load specified by AS1720.1-1997 as

$$Q_E = 1.1Q^* k_2 k_{26} k_{27} / k_1 \qquad 5.73$$

where the sampling factor k_{28} in equation 5.64 is replaced by a constant value of 1.1.

The value of k_{28}=1.1 results from the derivation, (Leicester 1987), that k_{COM}=0.9 in equation 5.21. This was obtained from a cost optimisation analysis that assumed a proof test to cost around 1/300 of the cost of rectification of an in-service failure. Which, again, is questionable as a general rule. Shown to be an unnecessary precaution, k_{COM} was subsequently omitted from the resistance equation specified in AS1720.1-1997, (equ. 5.48 this text), but was retained for proof testing. This extra 10%, together with the load effects reliability factor $\gamma_{S,typ}$=1.35 that is implicit in Q^*, raises Q_E arbitrarily, some 50% above the actual, unfactored, loading the structure is expected to sustain in service without regard to structure quality or structure importance.

It is evident from equation 5.72, that the factor 1.1 is the special case of the prototype test load factor k_{28} given by substituting n=1 and V_{prot}=0. Which gives γ_T=1.0 and the product ϕ_BI =0.9. The arbitrary nature of the factor is removed by substituting in equation 5.73 a true value of k_{28} obtained from properly assessed values for ϕ_B and I and by a more precise assessment of γ_S as applied to Q^*. When subject to an accurately known loading regime, γ_S=1.0.

5.8 SAMPLING FACTORS γ_M AND γ_T.

The factors γ_M, Leicester (1986), and γ_T, Leicester (1987), serve a common purpose. They compensate for the probability that the number of samples selected for test may be insufficient to characterize a population. For material / component characterization, the sample strength computed from the test data is reduced by γ_M when sample numbers are too low. Thus, for any structure designed with that particular characteristic strength, the load necessary to cause a failure is effectively raised to compensate for the characteristic's uncertainty.

The derivation of γ_M assumes a lognormal distribution and a value of 0.9 for k_{COM}. In developing the γ_M algorithm, the point was made, that, the "*principal aim was to achieve extreme simplicity in application*". The result is equation 5.25. For structure testing, the prototype test load, Q_E, is raised by γ_T thereby reducing the effective strength of the sample structure to offset the possibility that it may be unrepresentative of the population from which it is drawn.

The derivation of γ_T is based on a Weibull distribution which is slightly more conservative than log-normal for the range of variability, 0.1 to 0.4, and makes no reference to k_{COM}. The result is equation 5.70. Both equations 5.25 and 5.70 provide a confidence level of 75% that a 5% probability of failure or better will be achieved and "*their efficiencies are roughly comparable*", (Leicester 1986c).

Because both γ_M and γ_T are equally capable of performing the same statistical operation, the foregoing suggests that either is suitable for both material / component characterization and structure testing. The equation for γ_T is no more difficult to apply than that for γ_M and reference to Table 5.26 demonstrates that γ_T covers the 0.1 to 0.4 range of variability more realistically than γ_M^{-1}.

Replacing γ_M with γ_T^{-1} in equation 5.24 gives

$$R_{k,0.05true} = \left[1-\left(\frac{27}{n}\right)^{-V_M}\right] R_{0.05,sample} \qquad 5.74$$

Clarity is improved by employing the same factor in AS/NZS 4063-1992 and AS 1720.1-1997,

Table 5.26 Comparative Values of Reliability Factors γ_M^{-1} and γ_T

No. of units tested. n	Reliability factors	Coefficient of variation of sample			
		0.1	0.2	0.3	0.4
1	γ_M^{-1}	1.4	2.2	5.3	∞
	γ_T	**1.4**	**1.9**	**2.7**	**3.7**
2	γ_M^{-1}	1.2	1.6	2.3	4.2
	γ_T	**1.3**	**1.7**	**2.2**	**2.8**
5	γ_M^{-1}	1.2	1.3	1.6	1.9
	γ_T	**1.2**	**1.4**	**1.7**	**2.0**
10	γ_M^{-1}	1.1	1.2	1.4	1.8
	γ_T	**1.1**	**1.2**	**1.4**	**1.5**
30	γ_M^{-1}	1.1	1.1	1.2	1.3
	γ_T	**1.0**	**1.0**	**1.0**	**1.0**

5.9 EFFECT OF ENVIRONMENT ON CAPACITY FACTOR.

Strength resistance factors Πk account for variability in environment and structure / component configuration. A significant shortcoming in the current application of reliability theory to timber structures, as specified in AS1720.1-1997, is the limited attention given to the effects that variations in humidity have on strength and deformation. This is contrary to the conclusion reached in Chapter Four, that the duration-of-load factor k_1 varies significantly with humidity and thus affects reliability.

The strength limit state is calibrated to the performance of framing sized sawn timber, characterized under the standard test regime of 3-5 minute load duration, 20°C and 65%rh and assumes that the 50 year value of k_1 is 0.57. Standard test conditions fit the SE3 environment, for which Table 4.4 indicates that k_1 should have a value of 0.60. Thus, the calibration method is particular to structures situated in an SE3 environment. Consequently, to retain the same as-built reliability expressed in Table 5.22, adjustments to the value k_1 for other environments are necessary.

Because the material reliability factor, γ_M, is encompassed by f_0' within the resistance equations 5.56, values of γ_0 (equation 5.63) are valid for any timber or reconstituted wood-based material that is characterized using equation 5.24. Consequently, the values of γ_0 given in Table 5.22 will apply equally to structures with particleboard elements.

For prototype and proof testing, the effect of climate on the test structure is allowed for by inserting, in equations 5.64 and 5.73 respectively, appropriate values of k_1 drawn from Table 4.4

5.10 VARIATE DISTRIBUTIONS.

The mathematical basis for the foregoing assumed lognormal frequency distributions for $Xq/\overline{X}$ and V_X to be less than 30%. Table 5.27 shows that when V_R is small the difference between types of distribution have practically no impact on the computed values of the resistance load factors in view of the amount of statistical manipulation and the number of approximations employed in their derivation. This is evident, in particular, for structural components of particleboard for which the 5%ile values of V_R for characteristic properties range between 0.04 and 0.10 so that their $Xq/\overline{X}$ ratios differed by less than 3%, whether the distributions are normal, log-normal, Weibull or gamma. Accuracy of the derivation may be improved by considering the tails of the distribution curves below the 5%ile and above the 95%ile. While the form of the distribution may be ignored, the constants derived may still be in considerable error unless the sample size is sufficiently large. For example, equation 5.25, (Leicester 1986), assumes a log-normal distribution. When $V_R = 0.1$, a sample of 10 pieces yields a characteristic property that could only be obtained with around 150 pieces when V_R=0.4. AS/NZS 4063-1992 specifies minimum sample sizes and applies equation 5.24 to the distribution below the 5%ile with the coefficient of variation determined from the test data as a whole.

Table 5.27 5%iles and 95%iles (q%) of $Xq/\overline{X}$ for different distributions and V_X.

Distribution	$q\%$	$Xq/\overline{X}$				
		V_X=0.1	V_X=0.2	V_X=0.3	V_X=0.4	V_X=0.5
Normal	5	0.836	0.671	0.507	0.342	0.118
Normal	95	1.164	1.329	1.493	1.658	1.822
Lognormal	**5**	**0.845**	**0.708**	**0.591**	**0.493**	**0.411**
Lognormal	**95**	**1.172**	**1.358**	**1.552**	**1.750**	**1.945**
Weibull	5	0.817	0.647	0.498	0.374	0.275
Weibull	95	1.142	1.305	1.489	1.689	1.903
Gamma	5	0.841	0.695	0.561	0.436	0.342
Gamma	95	1.170	1.350	1.541	1.752	1.938

5.11 IMPLEMENTATION OF FINDINGS

To implement the foregoing findings, the various amendments and additions to materials characterization and structural design codes that appear to be necessary are summarized below.

5.11.1 Nomenclature and Definitions

Add the following terms and their definitions.

$\gamma_{S,op}$ typifies the reliability factors assigned to various types of load by

AS 1170.1-1989, and is assessed currently to have a value of 1.35.

I is the factor that classifies structure importance in terms of a reliability index.

ϕ_B is the factor that accounts for the quality of design and construction activity.

$\gamma_{0,3.0}$ is the overall as-built structure reliability factor when $\beta = 3.0$.

V_B is the coefficient of variation of the as-built structure.

V_M is the coefficient of variation of a material, component, or structure.

k_1 is the duration-of-load factor, currently assessed as 0.57 for timber and plywood.

Add definitions of design and construction grades A-D given in para. 5.6.6b.

5.11.2 Structure Resistance and Test Load Estimates

Substitute, $\phi_B \mathrm{I}$ for ϕ in resistance equations 1a and 1b

$$R^* = \phi_B \mathrm{I} \Pi k f_o' X$$

$$R^* = \phi_B \mathrm{I} \Pi k n Q_{k,j}$$

Amend prototype and proof test load equations 5.64 and 5.73 respectively as follows,

$$Q_E = Q^* k_{28} k_2 k_{26} k_{27} / \phi_B \mathrm{I} k_1$$

$$Q_E = Q^* k_{26} k_{27} / \phi_B \mathrm{I} k_1$$

and note that Q^* is based on an assumed value of $\gamma_{S,typ} = 1.35$, which may be adjusted to suit the degree of certainty that is attached to estimated load effects.

Substitute Tables 5.20 and 5.21, this text, for Table 2.5 AS 1720.1-1997.

5.11.3 Characterization

Add to the structural design code the sampling factor employed for material, component or member characterization,

$$k_{28} = \frac{(27/n)^{V_M}}{\phi_B \mathrm{I}}$$

Amend characterization equations as follows,

$$R_{BASIC} = k_1 R_k / \gamma_{0.3.0}$$

$$R_k = \frac{R_{0.05,sample}}{(27/n)^{V_M}}$$

and incorporate them in the design code.

5.11 4 Calibration of Working Stress to Limit States Design Methods

To explain its composition and facilitate periodic review as more reliability data is accumulated, add to the structural design code the equation that calibrates resistance estimates obtained by working stress design methods to those by limit states design methods.

$$f_o' = \left(\frac{\gamma_{S,typ}}{0.85 k_1 \gamma_{0.3.0}} \right) f_{BASIC}$$

The calibration factor is assumed currently to equal 2.95.

5.12 CONCLUSIONS

There is an evident need to clarify the role of the capacity factor in estimating by computation, or verifying by test, the in-service resistance of structures. This is achieved by dividing the capacity factor into two distinct parts. One part assigns four grades of quality to design and construction, the other, three levels of structure importance. Thereby, the independent role that each part plays in estimating structure resistance is clarified. This aspect of the findings is not material or structure specific and is applicable to structures and materials generally.

An anomaly was uncovered which revealed that structure resistance estimated by AS 1720.1-1997 is coupled to material or components characterized by AS/NZS 4063-1992. Unwarranted enhancement of 5%ile property values resulted. The amended resistance and characterization equations overcome the anomaly. Which "*uncouples*" action effects from materials and avoids the complication in design procedures referred to by Stevens (1975).

The review enables characterization procedures to be incorporated within the design code. The number of reference documents is reduced, eliminating one source of design error. The revision of any particular factor independently of any other is possible and ongoing editorial amendment is thus simplified. Two sampling factor equations are presently used for characterization in Australian codes. Adopting one of the two for all characterization, whether material, component or structure, provides consistency and improves clarity.

It was shown that the one-percentile characteristic stresses for small clear specimens are 25%, or one stress grade, lower than those derived as 5%ile values from in-grade evaluation. Characteristic stresses obtained from tests on small clear specimens are specified for several species by AS 1720.1-1997. Only in-grade evaluation will remove this statistical difference and provide comparable assessments of reliability. It is essential that all reconstituted wood-based products, particularly plywood, LVL and particleboard, be in-grade evaluated to improve confidence in published characteristic design stresses.

Cornell (1969) believed that reliability methods will *"force more critical examination (of uncertainty) by the individual designer and by the profession as a whole of the important problems remaining in structural design"*. Implementation of the results of this review will better enable an engineer to match estimates of the in-service reliability of structures to available material and human resources. Confidence in reliability based design methods will be improved, and to this extent, the criticism made by Gromala et al (1999) is assuaged.

CHAPTER SIX

CHARACTERISTIC PROPERTIES OF PARTICLEBOARD

6.1 INTRODUCTION

The research data reveals a thorough appreciation of the effects of the process and material variables on short-term mechanical properties. Reliable characterization of particleboard is therefore possible when it is manufactured to a sustained satisfactory quality.

In this chapter, the 5%ile characteristic mechanical properties of particleboard are determined from which values of short-term ultimate properties suitable for use with limit state structural design methods are proposed. Whilst the characterization is confined to 19*mm* thick particleboard manufactured in accordance with AS/NZS 1859-1997 for structural flooring applications, it is equally applicable to any random three-layer particleboard of similar structural quality, thickness and density.

6.2 SHORT TERM PROPERTIES

6.2.1 Error in Test Methods

Characterization of short-term properties of particleboard is generally carried out using specimens loaded normal to the surface in bending, much less frequently in tension or compression parallel to the surface. The effect of testing error and its bearing on evaluating mechanical properties of wood and reconstituted wood panels is not discussed widely in the literature. In work on hardboard, Kloot (1954) referred to the departure of actual stress at ultimate load from that assumed in bending tests by simple elastic theory. Among sources of error identified, were tension induced by friction between specimen and supports and effective span shortened as deflection increased, but no attempt to quantify relative effects was made. Given that Kloot was referring to material generally thinner than particleboard and of nominally uniform cross section, these matters would be complicated further by the density gradients in 3-layer particleboards. It is probable therefore, that much of the published data provides approximate assessments rather true characteristics.

6.2.2 Published Short-Term Properties

McNatt (1975) published the most complete set of data on mean values and coefficients of variation for commercial particleboard made in the USA at that time. Brynildsen et al (1976) published short-term strength (5%ile) and elasticity (30%ile) values for Scandinavian particleboard, but without supporting information. Whilst differences in level exist between the various property inter-relationships inferred by McNatt and Brynildsen et al, they are none-the-less of the same order. The latter, being proposed for structural design use, probably reflects some subjective judgements with respect to emphasis and order, while McNatt simply presents basic test data for a board seen as suitable for structural use in 1975.

Hanley et al (1985) characterized the short-term properties of all board manufactured in Australia with the load applied both normal to the surface and in-plane. This data enabled the Australian industry to agree on the lowest acceptable mean values for density, internal bond strength and moduli of rupture and elasticity in bending when loaded normal to the surface specified in AS/NZS 1859-1997. However, characteristic values for in-plane properties for compression, shear and tension were omitted. A complete set of minimum mean short-term ultimate values for structural particleboard, Type C5, was published in BSS 5669-1989.

Adam (1997) tested flooring grade particleboard board obtained from all Australian manufacturers in 1996. When loaded normal to the surface, similar values of short-term ultimate bending strength and elastic modulus were obtained whether determined by testing full sheets, half sheets or small standard specimens. Coefficients of variation were between 10% and 11% for small specimens as opposed to values as low as 3.6% for full and half-length boards. Short-term 5%ile ultimate bending strengths around 20*MPa* and mean values for modulus of elasticity approaching 4000*MPa* were obtained from standard small specimens. Corresponding full and half board values were some 10% to 20% higher.

Neither the density of the particleboard examined, nor laboratory humidity and temperature, were recorded by Adam (1997). Similar board tested in the same laboratory during the same period had a density of $680kg/m^3$ and the laboratory is air conditioned to $20°C$ and $65\%rh$. The test environment may therefore be considered to satisfy AS/NZS 1859-1997 so that Adam's mechanical properties may be compared with those obtained by others for $680kg/m^3$ board tested in the standard test regime.

McNatt (1986) used ASTM standard test methods to examine the relationships between tension and shear for four particleboards with different furnishes. Three were $11mm$ thick boards made with large particles, (waferboards), with a density of $670kg/m^3$ and, the fourth was a normal commercially manufactured, phenolic bonded, $19mm$ thick particleboard with a density of $715kg/m^3$. For all practical purposes, short-term ultimate in-plane tension and shear stresses were equal for all four boards. Consequently, their respective planar shear modulus were not too dissimilar either, ranging between 1400 and $1480MPa$ for the less dense waferboards and increasing to $1600MPa$ for the commercial particleboard and thus followed linear relationship to density. The three waferboards were nominally 10% stronger in compression, but much stronger, (230%), in tension than the normal 3-layer commercial board. Coefficients of variation for the waferboard stresses were about twice those of the normal particleboard, indicating a much less reliable structural materia and reflecting their differing furnish and sectional densities.

McNatt compared various standard bending test methods and found that values from all, while differing slightly, were generally linearly interrelated. The fixed interrelationship existing between all short-term mechanical properties of particleboard enables ultimate short term planar properties to be inferred with better than reasonable certainty from those measured normal to the surface. Planar tensile and shear strength and related elastic moduli assessed in this way were used to design the prototype wall beams that were successfully tested to establish their structural feasibility. To confirm the assumed design stresses, 19*mm* particleboard was fully characterized and realized coefficients of variation less than 10% and 5%ile property values that were similar to those specified by Brynildsen et al (1976). Short-term properties of particleboard from various sources are summarized in Table 6.1.

Table 6.1 Short Term Ultimate Mechanical Properties of Particleboard *MPa*

Source/thickness *mm*	S.G.	f_b'	f_c'	f_t'	f_{sp}'	E_b	E_c	E_t	G
M^c Natt 1975 19	0.65	16				3500			
Brynildsen,et al 9-13	0.80	20				4750	3800	3800	1650
16-19	0.66	18				3750	3000	3000	1250
22-25	0.56	16				3000	2400	2400	1000
SWH*(1976) 13-18	0.65	20				3500			
10	0.72	25				4000			
		20	14	7	0.25	5500			2750
Hanley et al (1985) 19		16	11	5.5	0.25	4300			2150
		12	9	4.5	0.25	3500			1750
BSS 5268 1996 6-19	0.72	24	18	11	3.2	3750	2750	2500	1100
AS/NZS 1859 12-33	0.65	20				3000			
Adam (1997) 19	0.68	20				3880			
Taylor (1998-01) 19	0.68	20	17	8	6	3850	2930	2890	

Subscripts, *b, c, t,* and *sp*, refer to bending normal, and tension, compression and shear in-plane respectively.

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6.3 DETERMINATION OF CHARACTERISTIC PROPERTIES

6.3.1 Material Characterized

Characterization of mechanical properties was performed on one grade of particleboard only, namely, flooring grade 19*mm* particleboard with a density of 680*kg/m*^{3} that conformed to AS/NZS 1859.1-1997.

6.3.2 Test Methods

Standard test methods are specified by AS/NZS 4266-1995, *Reconstituted wood-based panels-Methods of test*, for determining mechanical properties in bending normal to the surface only. The other tests employed for the characterization of mechanical properties were conducted as described below. To standardize the characterization of particleboard in Australia and New Zealand it will be necessary to add relevant test methods to AS/NZS 4266-1995. All tests were conducted in standard laboratory conditions, 20°*C* and 65%*rh*.

6.3.3 Characteristic Strength Properties

Test results were evaluated using the formula specified by AS/NZS 4063-1992, *Timber, Stress-graded, In-grade strength and stiffness evaluation*, as given by

$$f_0' = \left(\frac{1.35}{0.8(1.3+0.7V_M)}\right)\left(1-\frac{2.7V_M}{\sqrt{n}}\right)f_{0.05,data} \qquad 6.1$$

where V_M is the coefficient of variation of n specimens.

f_0' normalised characteristic strength appropriate to failure mode

$f_{0.05,data}$ test data 5%ile value.

It will be noted that the term $\left(\dfrac{1.35}{0.8(1.3+0.7V_M)}\right)$ in equation 6.1 raises 5%ile strength by around 25%, that is, by one stress grade. However, as discussed in detail in Chapter Five, this term is misapplied and creates an anomaly between material characterization per AS/NZS 4063-1992 and the design code, AS 1720.1-1997. Removing the term removes the anomaly from equation 6.1 so that

$$f_0' = \left(1 - \frac{2.7V_M}{\sqrt{n}}\right) f_{0.05,data} \qquad 6.2$$

Hence, characteristic 5%ile strength properties evaluated using equation 6.2 are those described as the "5%ile strength" in Table 6.2.

Table 6.2 Characteristic 5%ile Short Term Ultimate Strength, *MPa*, of Random 3-Layer 19*mm* Structural Particleboard 680kg/m^3 **at** 20°*C* ***and*** 65%***rh***.

Property	f_b'	f_c	f_t'	f_{sp}'
Test data 5%ile	20.8	17.4	8.6	6.7
$1-2.7V_M/\sqrt{n}$	0.96	0.98	0.94	0.91
5%ile strength	**20.0**	**17.1**	**8.1**	**6.1**
$1.35/0.8(1.3+0.7V_M)$	1.26	1.27	1.22	1.22
f_0'	25.2	21.7	9.9	7.4

Subscripts, *b, t, c,* and *sp*, refer to bending normal, and tension, compression and shear in-plane respectively.

6.3.4 Characteristic Elastic Properties

The characteristic values of Young's Modulus were evaluated using the formulae specified by AS/NZS 4063-1992, as given by

$$E_0 = \left(1 - \frac{0.7V_M}{\sqrt{n}}\right) E_{mean,data} \qquad 6.3$$

where V_M is the coefficient of variation of n specimens.

E_0 characteristic mean elastic property.

$E_{mean,data}$ test mean data.

The modulus of rigidity, G, was determined from

$$G = \frac{E_t}{2.4} \tag{6.4}$$

which assumes Poisson's ratio is 0.2, Moareas and Irie (1999).

Characteristic mean values of elastic properties are set down in Table 6.3.

Table 6.3 Characteristic Mean Short Term Mean Elastic Properties, *(MPa)*, of Random 3-Layer 19*mm* Structural Particleboard $680kg/m^3$ **at** $20°C$ **and** $65\%rh$.

Property	E_b	E_c	E_t	G
Test data mean	4200	3230	3080	
$1-0.7V_M/\sqrt{n}$	0.98	0.98	0.99	
True mean E_0	4100	3150	3050	1300

Subscripts, b, ,c, t, refer to bending normal, and compression and tension in-plane, respectively.

6.4 CHARACTERISTIC DESIGN VALUES

From the foregoing characterization, limit states design values for structural particleboard manufactured for flooring applications to AS/NZS 1859.1-1997 are proposed for grade PB20 in Table 6.4. Mechanical properties are directly proportional to density, Hunt (1976), Hanley et al (1985), McNatt (1986), a 5% increase in density improves mechanical properties 25% and visa versa. Corresponding values for two other stress grades of 19*mm* board, PB15 and PB24, have been inferred from PB20 values. Strength is arranged in rounded 25% increments and elastic properties in rounded 15% increments to fit the format employed for the timber and plywood stress grades specified respectively in Tables 2.4 and 5.1, AS 1720.1-1997.

Table 6.4 Characteristic Property Values for Random 3-Layer 19*mm* Structural Particleboard *(MPa)* for Limit States Structural Design Use at 20°*C* and 65%*rh*.

Structural Grade	Density kg/m^3	f'_b	f'_c	f'_t	f'_{sp}	E_b	E_c	E_t	G
PB16	650	16	12	6.0	4.8	3550	2650	2650	1100
PB20	**680**	**20**	**15**	**7.5**	**6.0**	**4100**	**3050**	**3050**	**1250**
PB25	720	25	19	9.5	7.5	4700	3500	3500	1450

Subscripts, *b*. *tp*, *cp*, and *sp*, refer to bending normal, and tension, compression and shear in-plane respectively. Strength are 5%iles, elasticity are means.

CHAPTER SEVEN

PARTICLEBOARD TO TIMBER JOINTS

7.1 INTRODUCTION

In timber structures, joints are primarily formed mechanically by the use of dowel-type fasteners acting in shear. In particular, the principal method of connecting sheet materials such as particleboard to timber is by nails or screws driven normal to the sheet surface. Characteristic strength values suitable for limit states design for different types of mechanical fasteners connecting different species of timber and plywood are specified by AS 1720.1-1997, but none are specified for connections to particleboard.

In this chapter, characteristic values suitable for the limit states design of particleboard-to-timber connections formed with nails driven normal to the surface are established by methods specified by AS 1649-1998, *Timber-Methods of test for mechanical fasteners and connectors-Basic working loads and characteristic strengths.* Nails larger than 3.75*mm* diameter, nails with deformed or coated shanks, screws, and punched steel nail plates, lie outside the scope of this study, as are bolts and coach screws, and the larger surface type fasteners such as split rings and shear plates.

Because particleboard is suitable for structural use only in situations where its average moisture content remains below 12%, connections to unseasoned timber with moisture content above 15% are not considered. Whilst field-glued jointing between seasoned material is feasible, it is most uncommon, because without a workshop that ensures clean, dry, temperate conditions and closely regulated and monitored curing, reliability is extremely uncertain. Consequently, because this study is aimed at widening the use of particleboard in commonly met field conditions using normal trade skills, adhesive connections are not considered.

Design methodology for mechanical joints draws on experimental work performed over a long period in the Forest Products Laboratory (FPL) in the USA during the 1930's, the outcome of some of which, concerning actual structures, is discussed in Chapter Five. FPL work was adapted for Australian timbers by the Council for Scientific and Industrial Research (CSIR), (Langlands and Thomas, 1939) and investigations by CSIRO have continued since. The design and development of trusses for housing, made with unseasoned timber and nailed joints, was undertaken by the Commonwealth

Experimental Building Station, (CEBS), in the immediate post war period, 1946-50. Values of nail strength specified by AS1720.1-1997 are empirically derived by best-fit equations to test data.

Joints between members are the most critical parts of structures in general, particularly timber structures. They are also the most unreliable parts. Joints fastened with bolts and ring connectors are difficult to inspect after assembly, exhibit a high variability in design and construction quality, and lacking ductility, fail catastrophically without warning. In the absence of sufficient data on behaviour to enable reliability theory to be applied, obtaining sound joints with sufficiently predictable strength for such connectors, depends on rigorous, detailed, joint design, supported by adequate testing and adequate field supervision, for which, at present, there is no substitute.

As for members, AS 1720.1-1997 makes no provision for the effect on structure quality of design and construction methods or for relating joint reliability to structure importance. Both are central to estimates of structure reliability. Fortunately, nailed connections are ductile, which ensures that failures from visually detectable excessive deformation only and, possessing low variability, may be treated probabilistically.
In this chapter, appropriate factors for the design of nailed connections are developed which account for these effects.

7.2 JOINT DESIGN

7.2.1 Design Equations

AS 1720.1-1997 classifies nailed connections that are subject to lateral shear loads, as distinct from axial loads, as Type 1 joints. Design resistance for direct loads is given by

$$R^* = \phi N_j = \phi k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \geq S^* \qquad 7.1a$$

and for in-plane moments by $$R^* = \phi N_j = \phi k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \left[\sum_{i=1}^{n} \left(\frac{r_i}{r_{max.}} \right)^{\frac{3}{2}} \right] r_{max.} \geq S^* \qquad 7.2a$$

Modifying equations 7.1a and 7.2a to meet the conclusions reached in Chapter Five is achieved by substituting the product $\phi_B \phi_I$ for ϕ, giving joint resistance

for direct loads as $$R^* = \phi_B \phi_I N_j = \phi_B \phi_I k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \qquad 7.1b$$

and in-plane moments as $$R^* = \phi_B \phi_I N_j = \phi_B \phi_I k_1 k_{13} k_{14} k_{16} k_{17} n Q_k \left[\sum_{i=1}^{n} \left(\frac{r_i}{r_{max.}} \right)^{\frac{3}{2}} \right] r_{max.} \qquad 7.2b$$

where N_j is the characteristic joint strength, i.e., resistance of a nail group.

Q_k characteristic fastener capacity =2.95 Q_{BASIC}.

k_1 duration of load factor for joints; a function of k_1 for members.

k_{13} 1.0 for nails in side grain.

k_{14} 1.0 for nails in single shear, 2.0 in double shear.

k_{16} 1.2 for nails driven through close fitting holes in metal side plates,

1.1 for nails driven through plywood or particleboard gussets,

1.0 otherwise.

k_{17} multiple nail reducing factor for Type 1 joints.

n number of nails.

r_{max}. maximum value of r_i.

r_i distance from the i^{th} nail to the centroid of the nail group.

Values of 1.1 for k_{16} and 1.0 for k_{17} are suitable for particleboard connections, as discussed in paragraph 7.6.

7.2.2 Calibration for Limit States Design Use

Joint strength is material-density and fastener specific, governed by the density of each component and the type, size and location of fasteners with respect to the end and edge of each component. Each type of fastener imparts a unique character to a joint by virtue of its particular load transfer mechanism that is reflected in the variability and strength of that particular joint. Joints made with small fasteners that each transfer a small load, exhibit the least variability and hence possess greater reliability than those made with larger less ductile fasteners, i.e., bolts and ring connectors with V_R=0.2-0.3.

Characteristically, the variability of nailed timber-to-timber joints is closer to 0.1 than 0.2 and, as shown by this study, the variability of nailed particleboard to timber joints is generally below 0.1. Connections made with nails are usually contained within a member sized to suit allowable axial and bending stresses and will exhibit reasonably predictable ductile behaviour with low variability.

Joints and members are treated as separate design elements within a structure. As with member characterization, the statistical distribution of nailed joint strength and deformability is generally assumed to be lognormal. Consequently, the mathematics that underpin the development of reliability theory for the design of members is also valid for joint design and the conclusions reached in Chapter Five with respect to the reliability of members apply equally to nailed joints.

With working stress design, the basic working load, Q_{BASIC}, of a fastener specified by AS 1649-1988 is derived as from 3-5*min* tests at 20°*C* and 65%*rh*. The test value, Q_{TEST}, is divided by a load factor, γ_j, that accounts for uncertainty in translating test methodology into real joints fabricated from structural grade timber under varying standards of workmanship so that

$$Q_{BASIC} = Q_{TEST}/\gamma_j \qquad 7.3a$$

For a selected probability of failure, the load factor, γ_j, is given three values, each depending on the particular test load being evaluated; the lowest of which refers to deformability, the others to ultimate strength. The test load, Q_{TEST}, has three values also. Two strength based values are given by equations 7.3b and 7.3c.

$$Q_{BASIC} = Q_{TEST,1}/2.0 \qquad 7.3b$$

where Q_{TEST} is the 5%ile of the ultimate test load,

and

$$Q_{BASIC} = Q_{TEST,2}/2.5 \qquad 7.3c$$

where Q_{TEST} is the mean of the ultimate test load.

A single deformability based value is given by equation 7.3d

$$Q_{BASIC} = Q_{TEST,3}/1.25 \qquad 7.3d$$

where Q_{TEST} is the 5%ile of the test load at 2.5*mm* slip.

The basic working stress safe nail strength loads, Q_{BASIC}, transformed characteristic values, Q_k^*, for limit states design use with AS 1720.1-1997. Values of Q_k^*, are obtained from the calibration equation specified in AS 1649-1998, $Q_k^* = 2.95Q_{BASIC}$, which corresponds to equation 5.30d for calibrating members.

7.2.3 Reliability Index for Laterally Loaded Nailed Joints

As discussed in Chapter Five, the calibration point at which working stress design outcomes are expected to meet limit states design outcomes is represented by a reliability index β=3.5 for components that exhibit a variability V_R=0.2.

Substituting γ_j for γ_R in equation 5.8c gives

$$\gamma_0 = \gamma_{s,typ}\gamma_j \qquad 7.4$$

hence

$$\gamma_j = \gamma_0/\gamma_{s,typ} \qquad 7.5$$

Equation 5.20b gives the overall as-built reliability factor $\gamma_{0,3.5} = 1.4 + 1.5V_R$ for β=3.5 and substituting V_j=0.2 for V_R results in an overall as-built reliability factor of $\gamma_{0,3.5}$=1.7. Substituting in equation 7.5 the overall as-built factor γ_0=$\gamma_{0,3.5}$=1.7, and the load effects factor $\gamma_{s,op}$=1.35, (Chapter Five), gives the reliability factor for nailed joint resistance as γ_j=1.26. For practical purposes this equals the value of γ_j=1.25 that calibrates Q_{TEST} to Q_{BASIC} in the deformability limit equation, 7.3d.

Thus, the probability of failure of nailed joints that deform 2.5*mm* matches that of primary members when designed to meet a reliability index of β=3.5. That is, nailed joints are calibrated for limit states design use to a deformability limit of 2.5*mm* as the limit state rather than the ultimate (strength) limit state, which, as discussed later, is appreciably greater. Consequently, the characteristic nail values specified for limit states design use in Table 4.1B, AS 1720.1-1997 are deterministic modifications per equation 5.30d of the working stress design values specified in AS 1720-1988.

On the basis of equations 7.3, the theoretical probability of failure of nailed timber-to-timber joints in AS 1720.1-1997, Table 4.1B, therefore approximates $p_F = 10^{-3}$. No values of nail strength or test load factors γ_j for other probabilities of failure are given in either AS 1649-1998 or AS 1720.1-1997. However, γ_j may be evaluated for the deformability limit state from equations 5.20 and results in values around unity for house framing and secondary structures (β=3.0, $p_F = 10^{-3}$) increasing to around 1.5 for essential service structures (β=4.0, $p_F = 10^{-5}$). Hence, nail values that satisfy reliability indices of β=3.0 and 4.0 may be determined from equation 7.3a by substituting an appropriate value of γ_j. Rounded values of γ_j for nailed joints with a maximum slip of 2.5*mm* in seasoned timber are given in Table 7.1, those that give a reliability index of β=3.5 as specified in AS 1649-1998, Table 4, are set down in bold type.

Table 7.1 Load Factors, γ_j, related to Reliability Index, β, for Nailed Joints in Seasoned Timber with Maximum Joint Slip=2.5*mm*.

Reliability Index, β	Load Factor, γ_j		
	5%ile load 2.5*mm* slip	Mean max load	5%ile max load
3.0	1.0	2.0	1.6
3.5	**1.25**	**2.5**	**2.0**
4.0	1.5	3.0	2.4

Values in bold ex Table 4, AS1649-1998

It is noted that the proportional relationship between the mean and 5%ile maximum strength values of the load factor assumes a lognormal distribution and variability around 0.15, which is appropriate for nailed joints.

7.2.4 As-built Capacity Factor ϕ_B for Laterally Loaded Nailed Joints.

The quality of design and construction methods affects the strength of joints in the same manner as it does members and the derivation of the design and construction factor, ϕ_B, developed for members in Chapter Five is equally applicable to Type 1 nailed joints.

Values of ϕ_B related to as-built reliability and Design and Construction Grades A-D, are given in Table 5.20. Definitions of Design and Construction Grades are given in Chapter Five.

7.2.5 Importance Factor I

As-built reliability of structures is affected by the individual reliability of all joints and members. This exposes the need to evaluate separately the variability of each joint component to arrive at structure that will exhibit a consistent theoretical probability of failure throughout by attaining simultaneously the selected value of β in all members and joints. Hence, values of the importance factor, I, for the selected strata of reliability, inferred by β and the target as-built variability, V_B, of a structure, are those given in Table 5.24.

7.3 TEST EVALUATION OF NAILED JOINT CAPACITY

7.3.1 Type of Connection

A range of characteristic nail values is established by testing a series of Type 1 joints between structural particleboard and two typical timber species in a seasoned condition using two different nail diameters. Timber components are a high density hardwood, E. marginata (Jarrah) and a low density softwood, P. radiata (Radiata Pine). The nails are 2.5mm and 3.15mm diameter, plain shank, low carbon steel, as specified in AS 2334, "*Steel nails-metric series*".

7.3.2 Test Method.

Tests were carried out in accordance with AS 1649-1998, which defines nails as Category A fasteners. Timber free of defects and nails were obtained from randomly chosen merchants. The 19*mm* particleboard was flooring grade made to AS/NZS 1859-1995. Mean moisture contents and mean densities of assembly components after conditioning 20°*C* and 65%*rh* are recorded in Table 7.2.

Table 7.2 Mean Moisture Content and Density of Timber and Particleboard.

Material	Joint group AS 1720.2	Density (kg/m^3)		Mean moisture content %
		Range: Table 1 AS 1649-1998	Specimen Mean	
Mixed Hardwood	JD3	600-745		
P. radiata	JD4	480-595	480	9.6
Particleboard	JD3	-	720	9.8
E. marginata	JD1	>940	980	9.6

Ten specimens of seven joint assemblies were fabricated to conform to AS 1649-1998. Five assemblies were timber-particleboard connections, two were made with timber components only. Each consisted of three pieces joined by two nails penetrating either two of the three pieces and thus loaded in single shear, or all three pieces and thus loaded in double shear. Joints were assembled with a gap of 0.8*mm* between adjacent components and nails were driven through holes predrilled to 90% of the nail diameter. Refer Figure 7.1 for details. Nails, 45*mm* long × 2.5*mm* diameter and 50*mm* long × 3.15*mm* diameter, were used for four timber and particleboard single shear assemblies. One double shear joint between particleboard and P. radiata was assembled with 75*mm* long × 3.15*mm* nails. The two timber assemblies were made with both diameters of nails.

7.3.3 Test Assemblies.

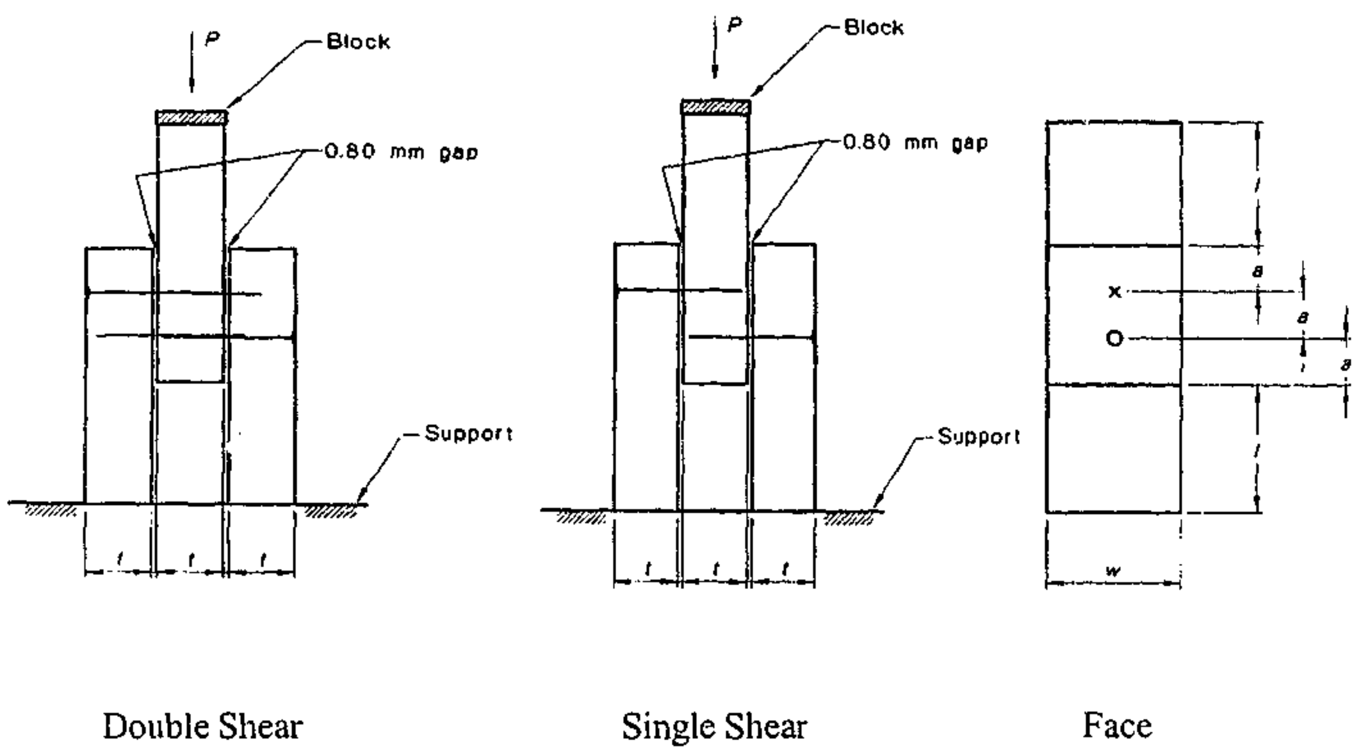


Figure 7.1 Single and Double Shear Joint Assemblies

Table 7.3 Dimensions of Joint Assemblies (*mm*).

Dimension	2.5*mm* dia. nails		3.15*mm* dia. nails	
	Timber	Particleboard	Timber	Particleboard
t	25	19	30	19
w	35	35	45	45
l	58	58	72	72
a	19	19	24	24

7.3.4 Evaluation of Test Data.

The mean and lower 5%ile values of the maximum load and the loads at joint deformations of 0.5*mm* and 2.5*mm* are determined for each assembly and their working stress design capacity evaluated as specified in Section 3, AS 1649–1998. Three unit lateral loads for each fastener, designated ULL_0, are computed, the lowest of which is taken as the basic lateral load per nail, Q_{BASIC}.

$$ULL_1 = Q_{TEST,1}/\gamma_1 = Q_{TEST,1}/4 \tag{7.6a}$$

$$ULL_2 = Q_{TEST,2}/\gamma_2 = Q_{TEST,2}/5 \tag{7.6b}$$

$$ULL_3 = Q_{TEST,3}/\gamma_3 = Q_{TEST,3}/2.5 \tag{7.6c}$$

where $Q_{TEST,1}$ is the lower 5%ile of the maximum load.

$Q_{TEST,2}$ average maximum load.

$Q_{TEST,3}$ lower 5%ile of the load at 2.5*mm* slip.

The load factors of 4 and 5 for γ_1 and γ_2, drawn from AS 1649-1998 for seasoned material, are based on CSIRO work c1975,. The value of γ_3 is given as 2.5 in AS 1649-1998, and values of $Q_{TEST,3}$ at 2.5*mm* slip are obtained from the test load / displacement curves.

7.3.5 Values of Nail Capacity from Tests

In light of the foregoing, the failure criterion for nailed particleboard-to-timber joints is based on a joint slip of 2.5*mm*. Characteristic lateral loads, Q_k^*, obtained using equation 7.3d and calibrated by equation 5.30d are given in Table 7.3 for 2.5*mm* and 3.15*mm* diameter nails. According to AS 1720.1-1997 and AS 1649-1998, characteristic lateral load capacities in single shear for other sizes are assumed to be directly proportional to the fastener diameter raised to the power 1.75. The values of Q_k^* obtained by test practically meet this ratio throughout which validates the interpolated and extrapolated values for 2.8*mm* and 3.75*mm* diameter nails also given in Table 7.3.

For other species of timber, Q_k^* is obtained by linear interpolation proportioned to the standard design densities given in Table 2 of AS 1720.2, *Timber structures: Part 2, Timber properties.* Similarly, for other densities of particleboard, Q_k^* may be obtained by linear proportion to 720kg/m^3, the mean density of the particleboard used for the test assemblies. It is noted that the densities of timber in the test assemblies, given in Table 7.2, lie near the low end of their respective Joint Group range. Resulting nail strength values are, therefore, close to respective minima for the Joint Group.

Table 7.3 Characteristic Limit States Single Shear Nail Capacity, Q_k^*, for 2.5*mm* Joint Slip. (Tests 3-5 minute duration at 20°*C* and 65%*rh*)

Test Joint Assembly	Q_k^* (*N*)	
	2.5*mm* diameter	3.15*mm* diameter
PR/PB/PR	630	1050
PB/PR/PB	990	1400
JR/PB/JR	1150	1700
PB/JR/PB	1300	1880
PR/PB/PR*	No test	1810
PR/PR/PR	970	1140
JR/JR/JR	1650	2760

*Double shear test. Extrapolated/interpolated values in italics.

Legend:
PR Radiata Pine (P. radiata).
JR Jarrah (E. marginata).
PB 19*mm* Flooring Grade Particleboard, AS 1859, density 720 kg/m^3.

Reference to Figure 7.1 will show that nails totally penetrated the particleboard component in both single and double shear specimens, thus representing the most common assembly anticipated for nail-jointed structures with particleboard panel elements. Consequently, values of Q_k^* determined by this test method may not be suitable for joints where nails do not pass fully through the less dense central core of particleboard. The effect of incomplete penetration was not investigated, but may lead to excessive long-term joint deformation and loss of strength, which invalidates the values in Table 7.3.

7.4 EMPIRICAL EVALUATION OF NAILED JOINT CAPACITY

7.4.1 Evaluation per AS 1720.1-1997

AS 1720.1-1997, Appendix C, *Joints in Timber Structures*, specifies empirical formulae for the characteristic strength of a nail in a Type A Joint in single shear as follows.

To limit joint deformation to Δ=2.5*mm*,

$$Q^{*}_{k,2.5} = 0.165 j_{13} h_{32} D^{1.75} \qquad 7.7a$$

where $Q^{*}_{k,2.5}$ is characteristic strength at 2.5*mm* slip, *N*

D nail diameter, *mm*

j_{13} =0.5 for seasoned timber loaded > 3 years

h_{32} =750 secant modulus / density factor, P. radiata, Joint Group JD4

Hence for JD4 timber $Q^{*}_{k,2.5} = 62D^{1.75}$ 7.7b

which for *D*=3.15*mm* gives $Q^{*}_{k,2.5}$ =460*N* / nail

and for *D*=2.5*mm* $Q^{*}_{k,2.5}$ =310*N* /nail

As given by AS 1720.1-1997, Table 4.1B, the limit states design values for 3.15*mm* and 2.5*mm* diameter nails at 2.5*mm* deformation in seasoned P. radiata, (JD4), is 810*N* and 545*N* respectively. Assuming k_1=0.57, these values reduce to 460*N* and 310*N* per nail for a duration-of-load of 50 years as given by equations 7.7.

To limit joint deformation to $\Delta \leq 0.5mm$ use is made of equation 7.8a.

$$\Delta = \left(\frac{44 j_{12}}{D^{3.5}}\right)\left(\frac{Q^{*}_{k,0.5}}{h_{32}}\right)^{2} \qquad 7.8a$$

where $Q^{*}_{k,0.5}$ is characteristic strength at $0.5mm$ slip, N

j_{12} =4 for seasoned timber loaded for more than 3 years

Substituting Δ=0.5mm and h_{32}=750 for JD4 timber gives

$$Q^{*}_{k,0.5} = 40D^{1.75} \qquad 7.8b$$

which for D=3.15mm gives $Q^{*}_{k,0.5}$=300N / nail.

and for D=2.5mm $Q^{*}_{k,0.5}$=200N / nail

7.4.2 Evaluation per Wilkinson (1972)

To gain additional insight into the effects of joining materials with dissimilar densities, and shed some light on the differing values obtained by test from AS 1649-1998 and those specified by AS 1720.1-1997, joint capacities are evaluated using an empirical formula developed by Wilkinson (1972). In common with AS 1720.1-1997 and AS 1649-1998, Wilkinson proportions the load / deformation ratio directly to $D^{1.75}$ so that the secant modulus is given by

$$\frac{P}{\Delta} = 0.5K\left(E^{0.25}\pi^{0.25}k_{0}^{\ 0.75}\right)D^{1.75} \qquad 7.9a$$

where
$$K = \frac{C(C+C^{3/4})}{2(C+C^{1/4})(C+C^{3/4})-(C-C^{3/4})^{2}} \qquad 7.9b$$

$$C = k_{02} / k_{01} \qquad 7.9c$$

P joint load, N

Δ joint slip, *mm*

E modulus of elasticity of nail, 2000*MPa*

D effective bearing width, (nail diameter), *mm*

k_{01} and k_{02} elastic bearing constants, *MPa/mm*

Values of k_{01} and k_{02} for joints made with smooth shank nails loaded perpendicular to the grain are computed by taking $k_0 = 348\rho_0$ from Wilkinson. Values of C are obtained by substituting for ρ_{01} and ρ_{02} the respective mean densities of the materials joined as shown in Table 7.1. Ratios of mean densities are given in Table 7.5.

Table 7.5 Ratios of Mean Densities.

Materials	PB / JR	PR / PB	PR / JR
Ratio of mean densities	0.74	0.66	0.49

Characteristic nail strengths for joint deformations less than 0.5*mm*, obtained according to AS1649-1998, are compared with those obtained from Wilkinson's formula and those specified by AS 1720.1-1997, Table 4.2A. Comparative values are given in Table 7.6.

Table 7.6 Comparative Limit States Characteristic 5%ile Capacities of a Nail in Single Shear for Type 1 Timber-Particleboard Joints with Maximum Slip=2.5*mm* at 20°*C* and 65%*rh*.

Joint Assembly	Q_k^* (*N*) 2.5*mm* diameter nail				Q_k^* (*N*) 3.15*mm* diameter nail			
	Wilkinson	AS 1649 2.5*mm* slip	AS 1720.1 Table 4.1B	AS 1720.1 Appendix C	Wilkinson	AS 1649 2.5*mm* slip	AS 1720.1 Table 4.1B	AS 1720.1 Appendix C
PR/PB/PR	930	890	-	-	1400	1500	-	-
PB/PR/PB	930	1270	-	-	1400	1490	-	-
JR/PB/JR	1200	1620	-	-	1800	2600	-	-
PB/JR/PB	1200	1830	-	-	1800	1880	-	-
RP/RP/RP	-	1070	545	310	-	1430	810	460
JR/JR/JR	-	2150	975	500	-	2710	1445	750

Legend:
PR Radiata Pine (P. radiata).
JR Jarrah (E. marginata).
PB 19*mm* Flooring Grade Particleboard to AS 1859, density 720 kg/m^3.

7.5 CHARACTERISTIC NAIL STRENGTH FOR LIMIT STATES DESIGN USE

The foregoing characterization indicates that the timber component controls joint strength when it has a density less than particleboard. However, when the timber component is the denser, test results indicate that strength is also influenced by the relative position of the particleboard in the joint. In light of the hiatus presented by the test results, further data is needed to determine appropriate design values for such joints and whether the associated design complication that arises, with its enlarged possibility of error, can be justified.

The single shear nail capacities specified in AS 1720.1-1997, Table 4.1B or when computed using Appendix C, are nominally half those obtained by test for 2.5*mm* slip from AS 1649-1998. However, values for 0.5*mm* slip given in AS 1720.1-1997, Appendix C, are reasonably consistent with the test values from AS 1649-1998 and with computed values from Wilkinson (1972). It appears that, in specifying values that are half those obtained by test, a question is raised over the reliability of the test methods to represent in-service joint behaviour. However, the low variability of the test results obtained in this study suggests that test results for the nail assemblies are reliable and that the specified nail values in AS 1720.1-1997, Table 4.1B are unduly conservative.

As discussed in Chapter 5, from a structural designer's standpoint, rather than specify reduced nail values, more clarity could be introduced into the design process by reducing the capacity factor, ϕ_B, to account for the implied unreliability of joints in service, that arises, generally speaking, from the normal neglect of the effects of secondary stresses and poor fabrication.

In the absence of better data on the reliability of joints in service, it is both practical and prudent to assume that nail strength has an upper limit given by mean particleboard density, which ranges between 650kg/m^3 and 720kg/m^3. That is, nail strength is limited to that given by JD3 material, which has a design density ranging between 600kg/m^3 and 745kg/m^3 (Table 1, AS 1649-1998, *Mean density range for joint groups.*).

In view of the uncertainty raised by the inconsistency referred to above, values specified for JD3 material in AS 1720.1-1997, Table 4.1B appear appropriate for nailed particleboard connections used in conjunction with the capacity factors discussed in Chapter 5. Characteristic nail values for the limit states design of structural particleboard-to-timber joints are given in Table7.6.

Table 7.7 Characteristic Nail Capacities (*N*) for Joints Between Random 3-Layer 19*mm* Structural Particleboard for Limit States Design

Structural Grade	Density kg/m^3	Characteristic Nail Capacity (*N*) Nail Diameter (*mm*)			
		2.5	2.8	3.15	3.75
PB16	650	690	840	1020	1400
PB20	680	720	880	1080	1470
PB25	**720**	**765**	**930**	**1135**	**1550**

Values in bold drawn from AS 1720.1-1997, Table 4.1B for JD3 timber.

7.6 ENVIRONMENTAL FACTORS.

7.6.1 Creep Factor

Nail capacities determined under standard test conditions, 20°C and 65%*rh* correspond to the SE2 environment. To design joints between seasoned timber and particleboard in service environments, SE0 and SE1, values of the creep factor may be obtained from Table 5.3 of Chapter Five.

7.6.2 Temperature Factor.

To adjust for the effects temperature, a reduction factor, (k_6 in AS1720.1-1997), may be obtained from Table 3.3, Chapter Three.

7.7 JOINT GEOMETRY

7.7.1 Spacing, End and Edge Distances.

Nail location with respect to the end and edge of the timber in a joint is governed by the tendency of wood to split, which is a function of tangential shrinkage and its low tensile strength perpendicular to the grain. Isotropic particleboard shows no similar tendency to split and virtually no timber shrinkage will take place in structures fabricated with seasoned timber. Therefore, provided nails are arranged within the minimum end and edge distance envelope and suitably spaced, loading orientation with respect to grain direction has no practical influence on the design strength of nailed timber-to-particleboard connections.

The flange / diaphragm connection in the prototype wall beam models built for this study were designed assuming a limit states capacity of 460*N* for 3.15*mm* diameter nails obtained from equation 7.7b. The models were fabricated using the spacing, end, and edge distances specified for timber connections by AS1720.1-1997, Table 4.4, *Minimum spacing, edge and end distances for nails*. Holes were not pre-bored. Reference to Chapter Eight will show that the nailed joints proved to be satisfactory, in that, when tested, all models maintained linearity to around 3×design load and reached an ultimate load around 4.8×design load. Joint geometry as specified by AS1720.1-1997 for timber and plywood is thereby demonstrated as being suitable for particleboard-to-timber connections.

7.7.2 Joint Reinforcement

No tests were conducted to examine the reinforcement offered by particleboard gussets because it is inferred that, in reducing the tendency of the timber component to split, the action of a plywood gusset is duplicated by a particleboard gusset. By virtue of their similar, nominally 50%, grain re-orientation, each layer of wood fibre in plywood or particleboard contributes a similar reinforcement across the grain, Hence, it is reasonable to assume that the 10% joint strengthening factor, specified by AS1720.1-1997 as k_{16}=1.1 for plywood gussets is equally valid for particleboard gussets. Similarly, and consistent with timber-to-timber or timber-plywood connections made with nails driven through close fitting holed steel plates, the basic strength of connections between such steel components and particleboard is raised 20%, putting k_{16}=1.2.

CHAPTER EIGHT

EXPERIMENTAL STRUCTURE

8.1 INTRODUCTION

The final objective of this study is to develop and evaluate a typical structural application in which particleboard acts as a primary element, namely a shear diaphragm in a long span heavily loaded wall beam. This chapter examines a typical structure, describes its design and construction and the testing undertaken to verify its structural adequacy.

The structure takes the form of a Vierendeel storey height wall beam with particleboard shear diaphragms, which is designed in accordance with AS 1720.1-1997 to support a limit states design dead load of 90*kN* over a span of 9*m* for a minimum of 50 years. The design is based on factors derived in Chapter Four for stress relaxation, in Chapter Five for the design and construction capacity factor, together with characteristic properties for particleboard and nail strength from Chapters Six and Seven.

Verification takes the form of testing several models, which demonstrates that the wall beam is capable of fully recovering from 3 × design load without damage and of supporting, ultimately, a test load in excess of 5 × design load.

The suspended timber framed floor is most commonly supported on stumps at around 1.5m centres both ways. Roof loads are also transferred through traditional wall framing to stumps. Substituting a long-span, storey-height beam that is capable of carrying roof and floor loads for a conventional load bearing wall, provides a means of reducing the number of supporting stumps to a minimum. Wall beams must satisfy various architectural requirements which, to permit circulation through the building, dictate the location and size of the shear diaphragms. They must also be capable of supporting the applied loads over a conventional 50 year service life.

One such wall beam, shown in Figure 8.1, spans 6*m* and supports 4*m* of lightweight roof and timber floor. It is 2.6*m* over-all height and consists of two, 2*m* wide, diaphragms composed of sheets of hardboard nailed each side of sawn timber flanges, and a central 2*m* opening. The structure has performed satisfactorily since its construction in 1964. On the basis of this structure and other experience, it was predicted that, by substituting particleboard for hardboard, a similar Vierendeel beam spanning 9*m* and consisting of three diaphragms with an opening each side of the central diaphragm was feasible.

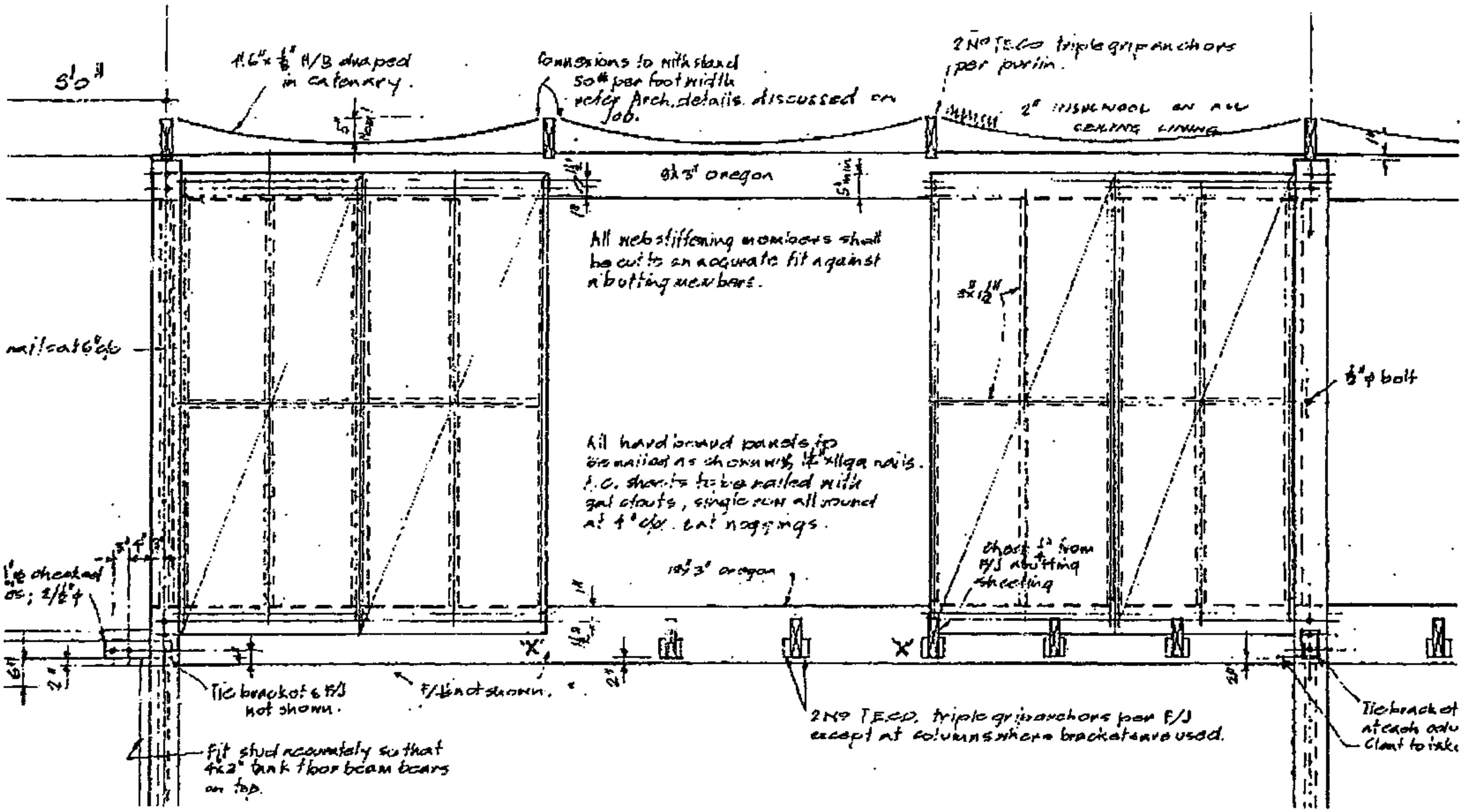


Figure 8.1 Wall Beam built in 1964

Particleboard is thus acting as a primary structural element, to test the feasibility of which, several modelled versions of the experimental wall beam structure shown in Figure 8.2 were designed and tested.

8.2 EXPERIMENTAL STRUCTURE

The experimental wall beam spans 9*m*, has an overall height of 2.7*m* and a clearance between flanges of 2.1*m* to permit access. The overall height suits a standard length of particleboard and the three diaphragms, which are each 1.8*m* wide, suit multiples of both 450*mm* and 600*mm* as commonly used in Australian domestic building practice. Diaphragms consist of two sheets of 2.7*m*×0.9*m* 20*mm* flooring grade particleboard conforming to AS/NZS 1859-1998, and are sandwiched between a pair of MGP12, P. radiata flanges. Diaphragm stiffeners are also MGP12, P. radiata.

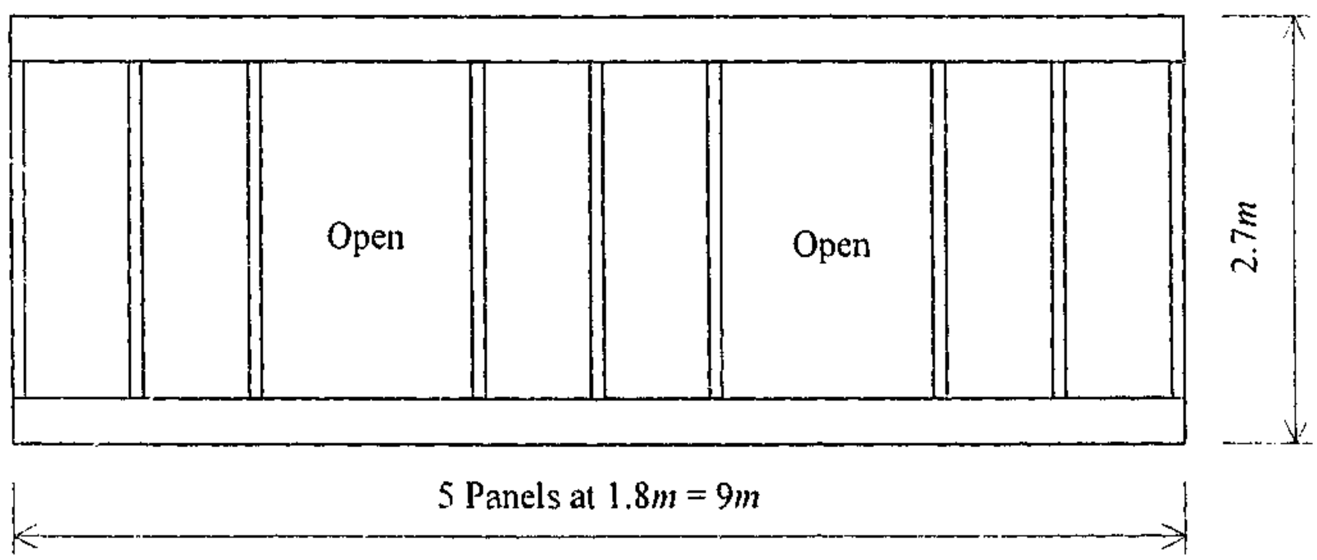


Figure 8.2 General Arrangement of Experimental Wall Beam

8.3 CHARACTERISTIC MATERIAL PROPERTIES

8.3.1 Particleboard

The diaphragms of the experimental structural model consisted of grade PB20 particleboard with the characteristic properties given in Table 6.4. Expressed in *MPa*, 5%ile strength properties are; f_b'=6.0, f_c'=4.5, f_t'=2.7, f_s'=2.3, and mean short-term moduli, E_b=4100, $E_c=E_t$=3050, G=1300. As discussed in Chapters Five and Six, the characteristic strength values are around 25% less than those obtained using AS/NZS 4063-1992.

8.3.2 Timber

The timber forming the flanges and diaphragm stiffeners is in-grade tested P. radiata, known commercially as MGP 12, with moisture content approximately 10%. The characteristic short-term strength properties adopted are those specified in AS 1720.1-1997, in *MPa*, as, f_b'=28, f_c'=29, f_t'=15, f_s'=6.5, and mean short-term elastic moduli as, $E_b=E_c=E_t$=12700, G=850.

8.3.3 Nails

As shown in Chapter Seven, to restrict joint slip in the model to a maximum of 2.5*mm*, the characteristic strength determined in accordance with AS1720.1-1997, Appendix C, for Type 1 connections in single shear to P. radiata were 460*N* for 3.15*mm* diameter nails and 310*N* for 2.8*mm* diameter nails.

8.4 MODIFICATION FACTORS

The following modifying factors form the product $\phi\Pi k$ in the resistance equation 1.1a. Except as determined below, values of other factors were taken as unity.

8.4.1 Capacity Factor ϕ_B and Importance factor I

The quality of design and construction of the experimental model, as defined in Chapter Five, was assumed to be Grade C, hence, ϕ_B=0.86. The beam was treated as a structural element with a Reliability Index β=3.0, hence I =1.0, their product, ϕ_BI, giving an overall capacity factor of 0.86.

8.4.2 Duration-of-Load Factors for 50 year Dead Loads

8.4.2.1 Particleboard

As determined by this study, Chapter Four, k_1=0.30.

8.4.2.2 Timber

As inferred by this study for structural timber, Chapter Four, k_1=0.48.

8.4.2.3 Nailed Timber-to-Particleboard Joints

As specified by AS 1720.1-1997, Appendix C, the duration-of-load factor is 0.50, which implies that displacements double in the long term under permanent load.

8.4.3 Load Sharing Factor

The load sharing factor, specified by AS 1720.1-1997 as having a value of 1.14, is applied to the twin timber components.

8.4.4 Stability Factors

8.4.4.1 Diaphragm

The critical buckling stress is determined in paragraph 8.6.2.

8.4.4.2 Flanges

Lateral and torsional restraint is applied to flanges at panel points and mid-panel points. Maximum effective length of the flanges is thus 1.8m for beam buckling and 0.9m for column buckling. Taking the material constant ρ_b=0.85, the stability factors, determined in accordance with AS 1720.1-1997, are k_{12}=0.65 for beam buckling, and k_{12}=1.0 and 0.65 for column buckling about the major and minor axis respectively.

8.4.4.3 Diaphragm Stiffeners

The clear distance between flanges, 2.1m, was taken as the effective length. Determined in accordance with equation 3.3(5) in AS 1720.1-1997, the stiffener buckling factor k_{12}=1.0.

8.5 MODEL WALL BEAM LOADING SYSTEM

The experimental models of the wall beams were designed to span 9*m*, and spaced at 4*m* centres to support a timber framed tiled roof and a timber framed floor. Limit states design loading was determined in accordance with AS 1170.1-1989, which specifies the estimated strength limit state load effect, S^* as

$$S^* = 1.25G + 1.5Q$$

As discussed in Chapter Four, by considering an appropriate portion of the floor live load, (Q), as dead load, (G), the recoverable transient effects of live loads on floor and roof may be neglected in designing timber structures. As shown subsequently in Section 8.7, the effect of this assumption was to magnify the test load and was, therefore, conservative.

Spacing the wall beams at 4*m* centres, and taking 40% of the floor live load of 1.5*kPa* as permanent load, the limit state design load due to the floor and wall was

$$Q^*_{FW} = 4[1.25(0.5 + 0.4 \times 1.5)] = 5.5\,kN/m$$

and due to the tiled roof and ceiling

$$Q^*_{RC} = 4[1.25 \times 0.9] = 4.5\,kN/m$$

making a total permanent limit states design load of Q^*_F=10*kN/m* or 90*kN* on the full scale 9*m* span

Roof and ceiling apply a factored dead load of 4.5*kN/m* to the top flange while the floor and the wall apply a factored dead load equivalent of 5.5*kN/m* to the bottom flange. The beam was divided into five 1.8*m* panels. It is reasonable to regard the outer half of the load in each end panel as applied directly over a support, so that the load system reduces to one consisting of four sets of two concentrated loads applied at the inner panel points. That is, 8.1*kN* at each top flange panel point and 9.9*kN* at each bottom flange panel point as shown in the design loading diagram, Figure 8.3.

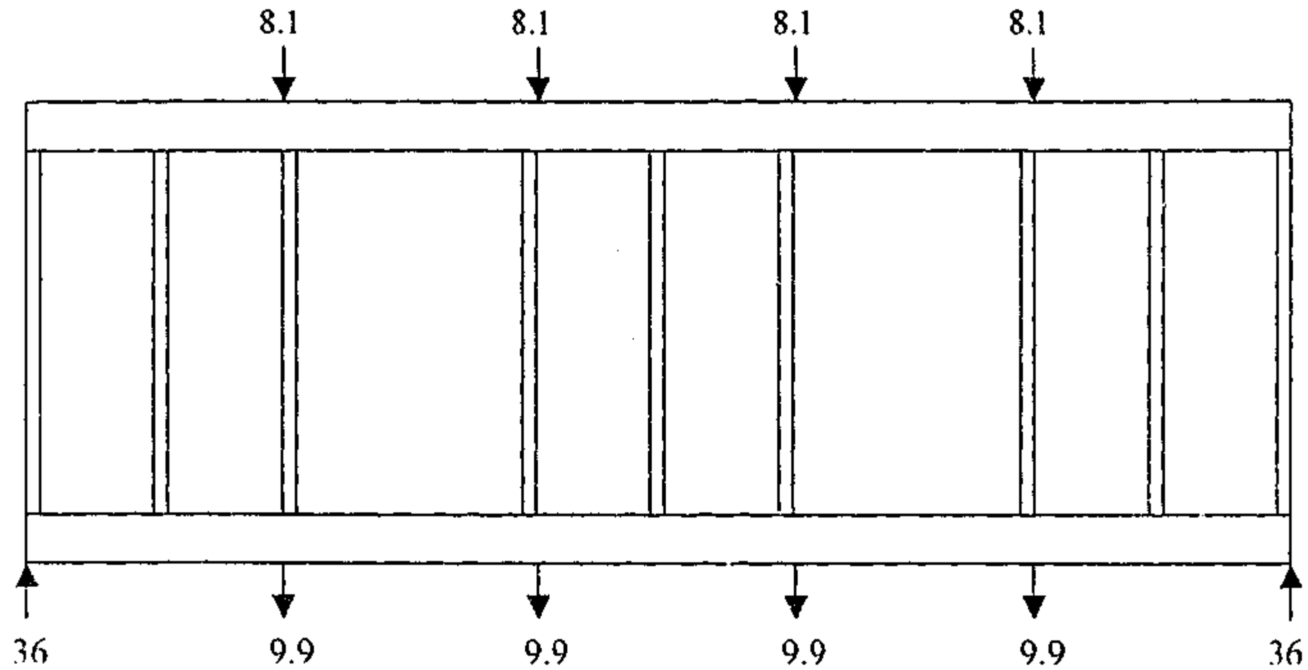


Figure 8.3 Design Loading for Experimental Wall Beam (Q_F^* kN)

8.6 STRUCTURAL DESIGN

8.6.1 Design Methodology

The design of the beam is based on accepted structural principles and elastic theory. Design assumptions are discussed as they arise and methodology for determining resistance follows AS 1720.1-1997 as expressed by design equations 1a and 1b,

for members $\phi R = \phi \Pi k f_0' X \geq R^*$

for joints $\phi R = \phi \Pi k n Q_k \geq R^*$

8.6.2 Diaphragms

The diaphragm height is nominally 2100*mm* clear and its width 1650*mm* clear between perimeter members. It is composed of two panels that are joined by nails to a vertical timber member located at the centre of the diaphragm. From Timoshenko (1947), the critical buckling stress, σ_{cr}, that a rectangular panel can withstand, when simply supported on four sides and subject to uniform shear, is given as

$$\sigma_{cr} = \eta\sigma_E \tag{8.1}$$

$$\sigma_E = \frac{\pi^2 E t^2}{12b^2(1-\mu^2)} \tag{8.2}$$

where E is the elastic modulus

t plate thickness

μ Poisson's ratio=0.2

The coefficient $$\eta = \left(\frac{a}{mb} + \frac{mb}{a}\right) \tag{8.3}$$

where m number of buckling waves into which the plate divides

a plate length

b plate width

Putting a/b =2.6 gives the coefficient η=6.2. Therefore, the critical short-term buckling stress of a 19*mm* thick particleboard plate with E=4100*MPa* is σ_{cr} = 16*MPa*, which exceeds its 5%ile characteristic compressive strength, f_c' =4.5*MPa*, indicating that the panels will not buckle before failing in compression.

The design load effect shear force, V_F^*=36*kN*, is transferred to the diaphragm at the rate of approximately 17*kN/m* acting around its perimeter. Putting $\phi \Pi k$ =0.26 in equation 1.1a, the shear capacity per meter of a 19*mm* thick particleboard diaphragm is

$$\phi V_F = (0.26 \times 6.0)A = 29kN/m$$

which exceeds the applied perimeter shear force.

8.6.3 Flanges

The experimental structure is statically indeterminate. An approximate analysis is rendered possible by assuming points of contraflexure to exist in the middle of the flanges crossing the open panels at mid-length. Because flanges are nominally of equal stiffness, it was assumed that the shear force acting across an open panel was shared equally between them. It is also assumed that long term shear deflection across the open panel is not greater than 15*mm*, that is, half the overall beam deflection of span / 300. On this basis, design load effects create, in each flange, a bending moment, $M_F^* = 8.1 + 0.5 = 8.6kNm$, an axial load, N_F^*=34*kN* and a flexural shear force, V_F^* =9*kN*.

Assuming that the flanges consist of a pair of 290×45 MGP12 members, the combined section properties of which are, $Z = 1.26 \times 10^6 mm^3$ and $A = 26000mm^2$.

In the upper flange, the load/capacity interaction ratios for bending and axial compressive stress using AS 1720.1-1997 indicate that the flange is some 10% under stressed and in the lower flange for bending and axial tensile stress, some 25%. Computed flexural shear capacity indicates that the flanges are more than adequate.

8.6.4 Diaphragm Stiffeners

Stiffeners consisting of a 70*mm* × 45*mm*, A=6300mm^2, are fitted either side of the diaphragm at each panel point. It is assumed that the whole beam limit states design load effects reaction of 36*kN* is transferred through the timber to the diaphragm. The computed stiffener compressive capacity is well in excess of the reaction force

8.6.5 Diaphragm Connection to Flanges

The connection between the twin flange members and the diaphragm was formed by driving 3.15*mm* diameter nails through both flanges and particleboard so that they acted in double shear. Half their number were driven through the near flange, passing through the particleboard and penetrating the far member of the flange, the other half from the far side. As discussed in Chapter Seven, this arrangement was tested in accordance with AS 1649-1998 and found to have a capacity of 1810N / nail in single shear. However, in view of the questions raised, it was decided that a design nail strength of 460N / nail in single shear, computed in accordance with Appendix of AS 1720.1-1997 for P. radiata, (species group JD4), would be adopted for the test model. Hence, joint design capacity, $N_j = 0.86(460 \times 2) = 800N$ / nail for 3.15*mm* diameter nails in double shear. Therefore, to transfer a shear force of 36*kN*, 45 such nails were required.

This joint strength includes a provision for long-term deformation and assumes that a long-term maximum nail slip of 2.5*mm* could ultimately develop, which, in view of the test results, (Chapter Seven), would more probably not exceed 1.3*mm*. During a 15 minute load test, it is probable that a slip less than 0.5*mm* will develop at design load increasing to 2.5*mm* under full test load.

8.6.6 Diaphragm Connection to Stiffeners

Stress concentrations develop at each stiffener junction with the flanges. At the lower panel points, a tensile force was transferred from flange to diaphragm, at the upper a similar compressive force. Experience indicated that hardboard under long-term stress would probably transfer the load to the diaphragm without reinforcement, but it was uncertain that particleboard could do so. The first truncated model was tested without reinforcement, because a theoretical analysis of the distribution of stress in the vicinity is uncertain and would need confirmation by test. In the event, this model failed at 4.7 × design load, which exceeds 90% of the ultimate test load, which, from a practical standpoint, is acceptable. However, all models were subsequently strengthened at each panel point to transfer 50% of the shear force directly to the stiffeners through commercial "nail-on" steel plates to ensure that failure would be governed by diaphragm action.

8.7 LOAD TEST

8.7.1 Prototype Test Loading for Dead Load Effects

Referring to equation 5.64, the test loading, Q_E, for a prototype structure is

$$Q_E = k_{28} Q^* k_2 k_{26} k_{27} / k_1$$

or the test load / design load ratio is $\dfrac{Q_E}{Q^*} = k_{28} k_2 k_{26} k_{27} / k_1$ 8.4

To distinguish between the particular values of Q^* and Q_E that pertain to each of the wall beam models that are examined, the following suffixes are used:

Q_F^*, Q_{EF} Full scale $9m$ beam model

Q_T^*, Q_{ET} Truncated model

$Q^{*}_{1/3}$, $Q_{E1/3}$ One third model

Q^{*}_{M} Finite element mathematical model

The product of $k_2k_{26}k_{27}$ is unity, whereas values of k_{28} and k_1 relate to the particular component material. As discussed in Chapter Five, the factor k_{28} accounts for the variability in the population of structures that the tested prototype structure represents. Values of k_{28} are given by equation 5.72 as

$$k_{28} = \frac{\left(\frac{27}{n}\right)^{V_R}}{\phi_B \mathrm{I}}$$

and are listed in Table 8.1 for the three materials that compose the model beam. Values of the duration-of-load factors, k_1, for permanent load effects, as developed in Chapter Four and referred to in paragraph 8.4.2 for particleboard and sawn timber, are also listed in Table 8.1.

Substituting the appropriate values of k_1 and k_{28} in equation 8.4 results in values of $\frac{Q_{EF}}{Q^{*}_{F}}$ for the limit states design load effect that includes 40% of the floor live load specified by AS 1170.1-1989. Values of $\frac{Q_{EF}}{Q^{*}_{F}}$ are given in Table 8.1.

Table 8.1 Prototype Test Loads Q_{EF} on Full Scale Experimental Beam Model for Dead Load Effects Q_F^*

Component	$\phi_B 1$	V_R	k_{28}	k_1	Q_{EF} (kN)	$\frac{Q_{EF}}{Q_F^*}$
Particleboard	0.86	0.10	1.5	0.30	450	5.0
MGP 12	0.86	0.30	2.5	0.48	470	5.2
Nailed joints	0.86	0.15	1.7	0.50	315	3.5

To ensure that failure occurs first, theoretically, in the particleboard, the number of nails is raised in the ratio $\frac{450}{315}$, which increases to 64 the number required on each side of the diaphragm, in lieu of the 45 computed to effect the shear transfer. This raises $\frac{Q_{EF}}{Q_F^*}$ to 5.1.

8.7.2 Prototype Test Loading for Live Load Effects

A different test load is specified by AS 1720.1-1997 for structures subject to combinations of dead and live load. Spacing the wall beams at $4m$ centres, the limit state design load due to floor and wall dead load and a floor live load of $1.5kPa$ as specified by AS 1170.1-1989 was

$$Q^*_{F,floor} = 4(1.25 \times 0.5 + 1.5 \times 1.5) = 11.5\,kN/m$$

and due to the tiled roof and ceiling

$$Q^*_{F,roof} = 4(1.25 \times 0.9) = 4.5\,kN/m$$

making a total limit states design load of Q_F^* =$16kN/m$ or $144kN$ on the full $9m$ span.

Referring to equation 5.64 for obtaining the test load, Q_{EF}, values of $\phi_B\phi_I$ and hence, k_{28}, remain unchanged, the value of k_1 specified for a medium term loading period by AS 1720.1-1997 rises to around 0.9 for timber and 0.6 for particleboard, and around 0.7 for nail joint stiffness.

Table 8.2 Test Loads on Experimental Beam for Limit States Live Load Effects

Component	$\phi_B\phi_I$	V_R	k_{28}	k_1	Q_E (kN)	$\frac{Q_E}{Q^*}$
Particleboard	0.86	0.10	1.5	0.60	360	2.5
MGP 12	0.86	0.30	2.5	0.90	400	2.8
Nailed joints	0.86	0.15	1.7	0.70	350	2.4

8.7.3 Prototype Test Loading, $5\,Q_F^*$, for Experimental Model Wall Beam

In all cases, the values of Q_{EF} determined for a combination of dead and live load, as shown in Table 8.2, are significantly less than those determined for dead load plus permanent live load given in Table 8.1, which justifies the omission of true transient floor live load in determining Q_{EF}. The omission of roof live load of 13.5 kN is similarly justified, in that, its addition is unable to raise Q_{EF} above the values in Table 8.1. Therefore, the test loading to be satisfied by the diaphragms in the experimental wall beam model is $5\,Q_F^*$=450kN as shown in Figure 8.4.

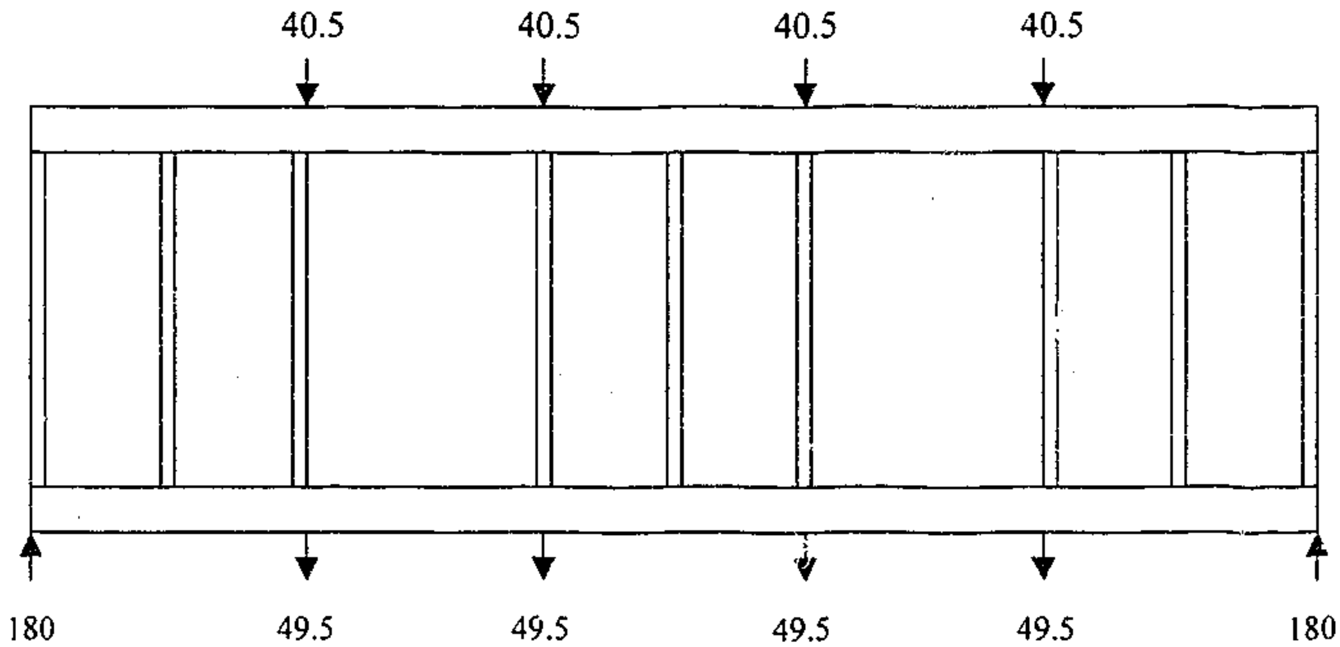
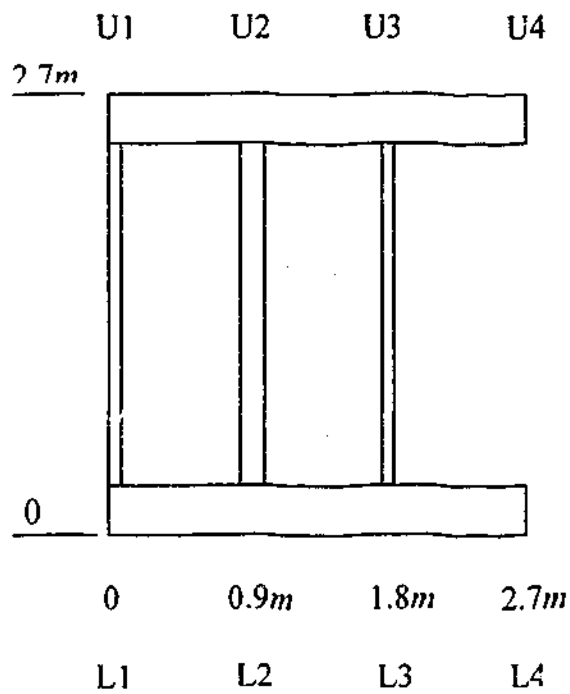


Figure 8.4 Load and Reactions on Experimental Model Wall Beam for Prototype Test ($5\,Q_F^*$ *kN*)

8.7.4 Laboratory Restrictions

Restricted laboratory space, and the possibility that the load capacity of the model could exceed 450*kN*, prohibited a full 9*m* wall from being tested. In order to reduce both the test load and the floor space occupied, the model was truncated as shown in Figure 8.5. The loading arrangement was modified to suit the truncation which halved the test load as shown in Figure 8.6.



Flanges MGP12		2 - 290×45
Stiffeners MGP12	Edge	2 - 70×45
	Middle	2 - 120×45
Diaphragm P-board Grade PB20		19 thick

Figure 8.5 Truncated Model - Structural Details and Reference Marking

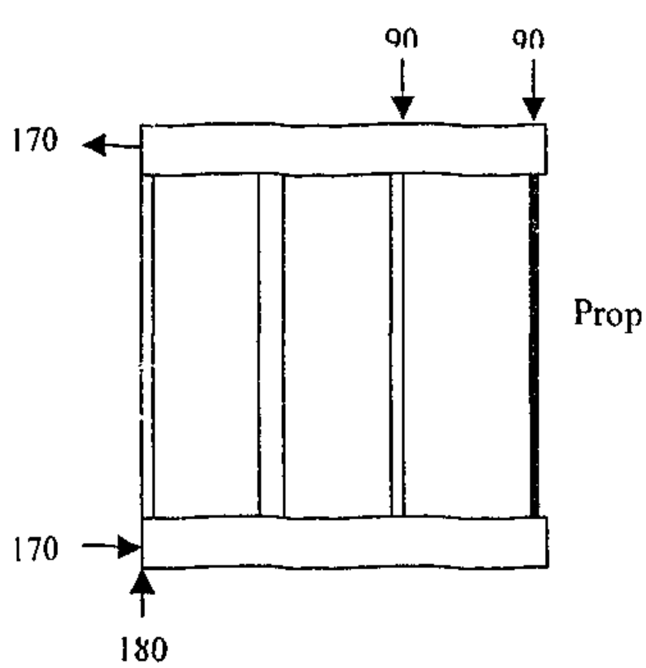


Figure 8.6 Test Loading on Truncated Model ($5\,Q_T^*$ kN) representing

Prototype Test Load on Full Scale Model Beam ($5\,Q_F^*$ kN)

8.7.5 Truncated Model Fabrication

Two truncated models were fabricated under field conditions by a carpenter using normal hand-held, low powered tools, and transported to the Monash University structures laboratory at Caulfield campus for testing. The steel plate strengthening at panel points was added in the laboratory.

8.7.6 Instrumentation

Transducers are positioned externally to record perimeter movement of the model as a whole and the overall diagonal movement in the diaphragms To obtain the stress distribution within the model, 60° strain rosettes are fixed to the diaphragms in six locations. All instruments are connected to an electronic data logging system that records load and deformations simultaneously at two second intervals.

8.7.7 Test Environment

Testing was performed the SE1 service environment referred to in Table 3.4, i.e., a ruling 20°*C* and 65%*rh* rising very occasionally to 80%*rh* for short periods when laboratory doors were opened to receive or remove material.

8.7.8 Test Method

The truncated model was mounted horizontally on a steel bed frame bolted to the laboratory reaction floor as shown in Figure 8.7.

The test load was applied through a hydraulic ram delivering the load at a uniform rate so that the end of linearity was reached between 3 and 5 minutes and the full test load,

$Q_{ET} = 5\,Q^{*}_{ET}$, around 2 minutes later. The ram force was applied to the top flange at two points through a spreader. One of the points of load application was placed at the mid-length of the top flange crossing the open panel, under which point a stiff prop was fitted to transmit load to the lower flange at its mid-length. The other loading point was aligned with the edge of the diaphragm. To reduce local crushing, steel plates were placed to beneath the active load points. The lower reaction is transmitted via a cement mortar plug, cast in-situ between the steel bed frame and the chamfered corner of the lower flange and the upper flange reaction to the bed frame via shear plates located on its gravity axis.

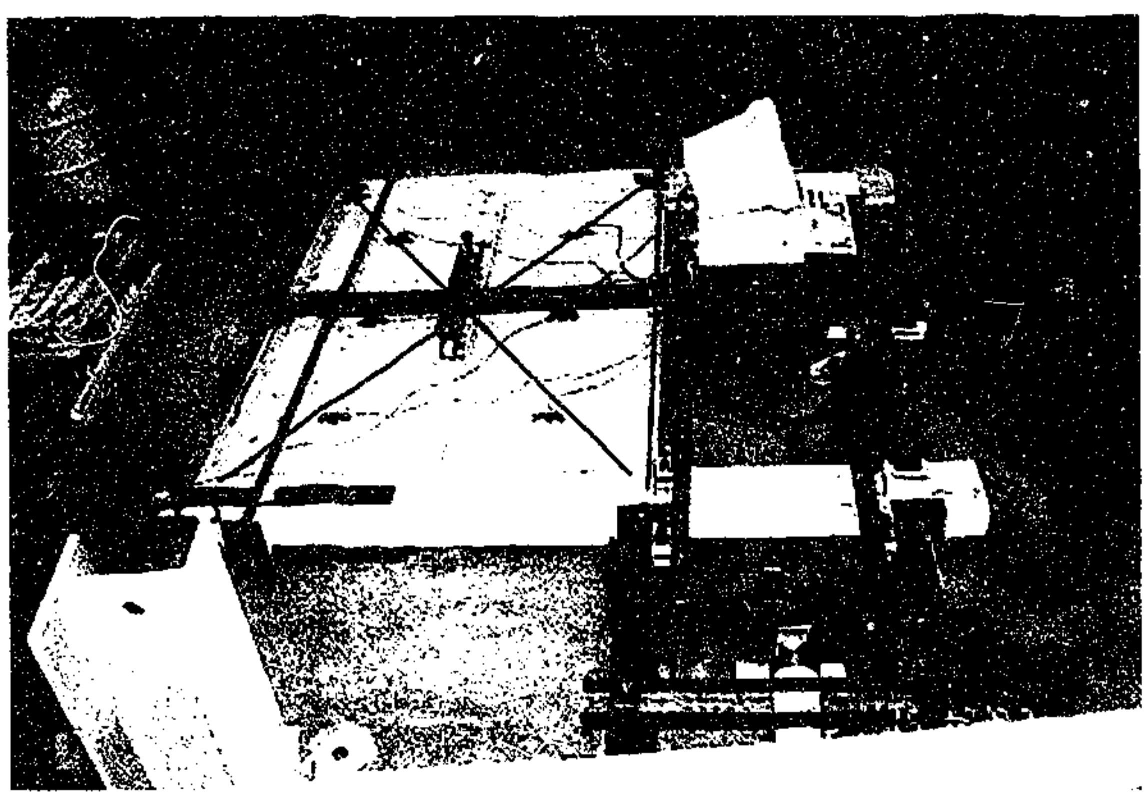


Figure 8.7a Truncated Model Test Rig Set-Up (viewed from upper flange)

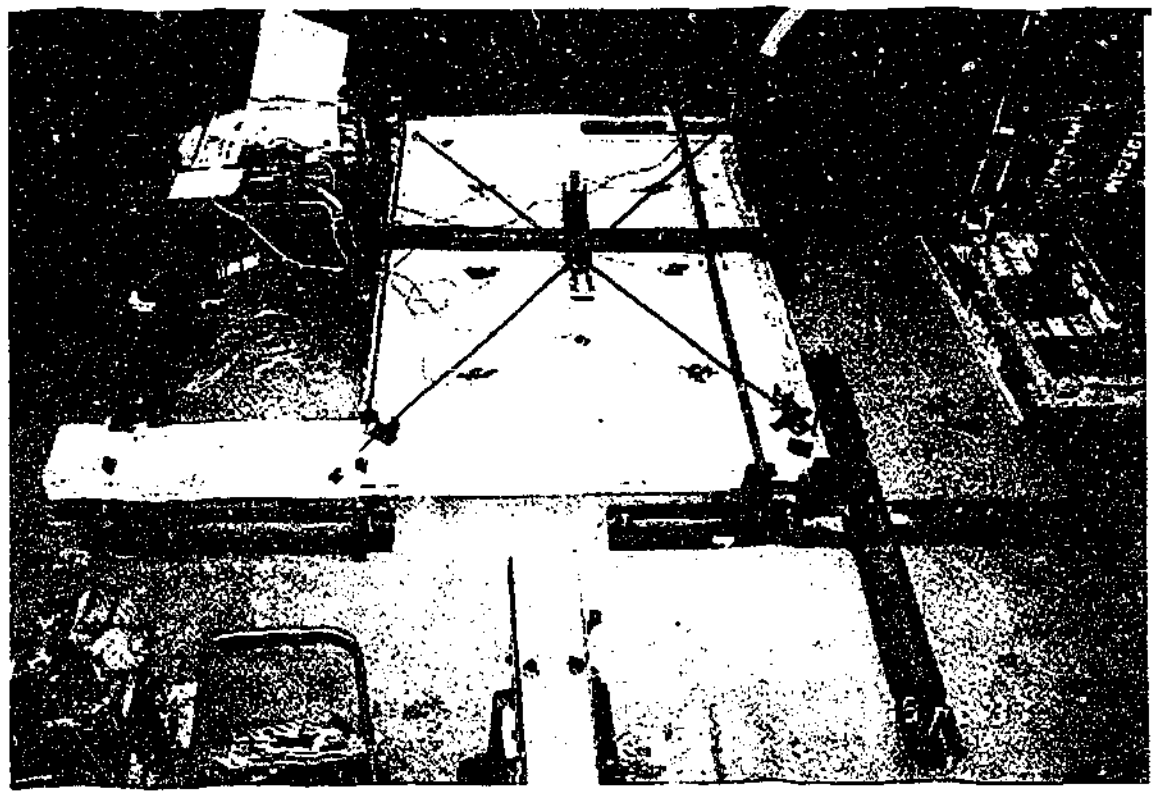
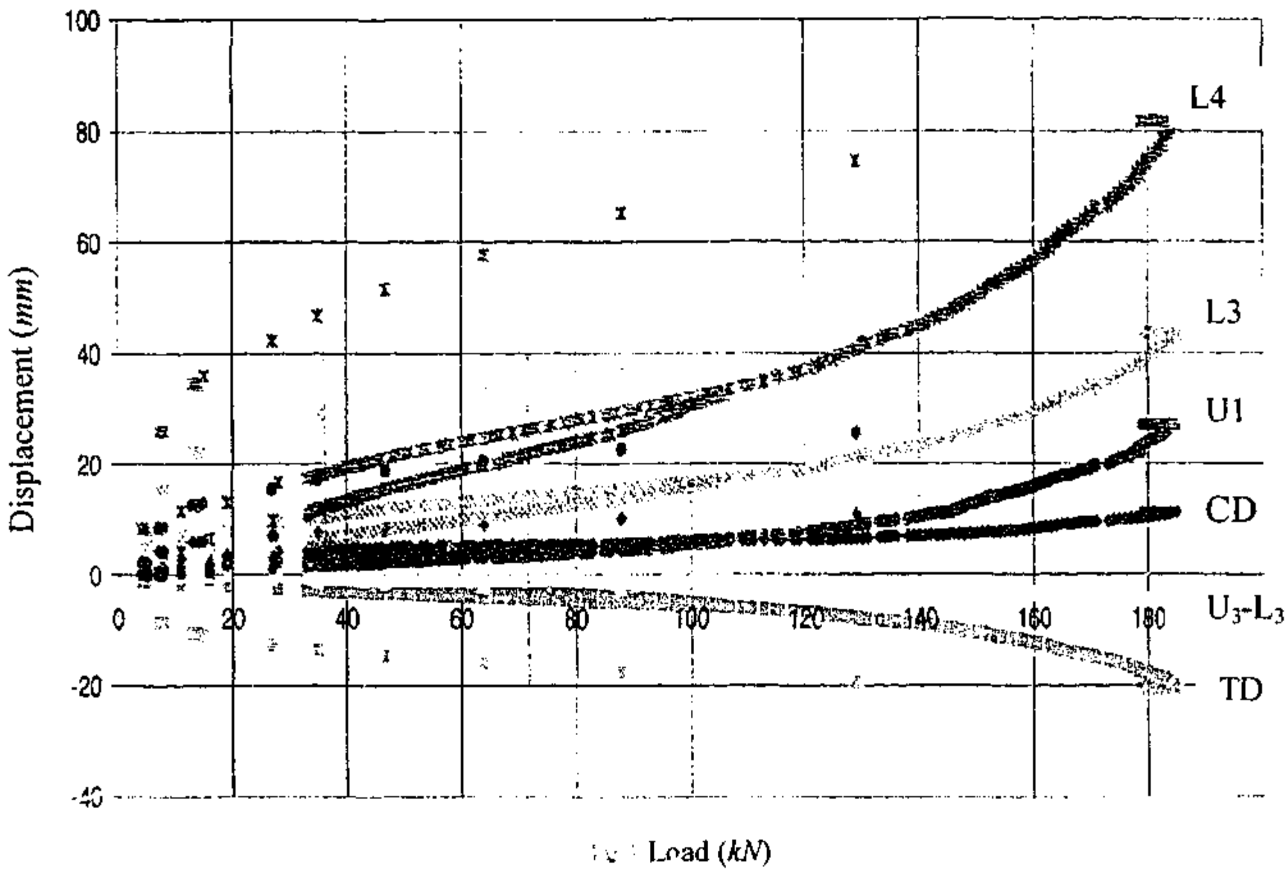


Figure 8.7b Truncated Model Test Rig Set-Up (viewed from lower flange)

The complete model was supported on rollers beneath the flanges and stiffeners at panel points and mid-panel points and under stiffeners at mid-height. To prevent buckling as the test load approaches $5Q_T^*$, restraint was effected by placing rollers above and below the diaphragm stiffeners at mid-height. Before the test began, the model was bedded into the reaction frame by applying a load of $1.5Q_T^*$ several times, allowing a recovery period not less than six times the loading period between each application to satisfy the ratio that permits a full recovery of deformation (Grossman and Nakai 1987).

8.7.9 Deformation of Truncated Model



Legend (Reference marking shown on Figure 8.5)

TD/CD	displacement along Compression/Tension diagonals in diaphragm
L3, L4	vertical displacement of lower flange.
U1	horizontal displacement of upper flange
U3-L3	relative displacement between flanges

Figure 8.8 Deformation of Truncated Model

To correctly model the left hand half of the full scale wall beam, the model should be supported vertically at L1, and horizontally at U6 and L6. At U6 and L6, no horizontal movement occurs because the full scale frame and the imposed load is symmetrical. But in the truncated model, the horizontal supports at U6 and L6 are located at U1 and L1.

The effect of this on vertical displacement at L3 and L4 is negligible, given that the axial stresses in the upper and lower flanges are in the order of $1MPa$ which results in a decrease in length between U1 and U6 of less than $0.2mm$ and a similar increase in length between L1 and L6. The true vertical displacement of the lower flange of the truncated model is thus closely approximated by subtracting the appropriate proportion of the horizontal movement at U1 from the vertical displacements at L3 and L4. Resulting displacements are given in Table 8.3.

Table 8.3 Lower Flange Deflection at 2.5 × Limit States Serviceability Design Load

Beam Model	Deflection (*mm*) at Panel Points and Mid-Panel Points					
	L1	L2	L3	L4	L5	L6
Truncated	0	-	10	18	-	-

Perimeter deformations and diaphragm strains remained linear with load to at least $3Q_T^*$. On reaching $185kN$, which is $5.1Q_F^*$, and slightly in excess of the equivalent prototype test load on the $9m$ span full scale wall beam, Q_{EF}, the test load was removed. When readings were taken several days later, deformations had all but completely recovered. Clearly the truncated model resisted an equivalent full scale test load satisfactorily, but the question arose. How closely did the method of applying the load represent the actual loading condition?

By reference to the test load arrangement on the truncated model, shown on Figure 8.6, it will be observed that, while correctly dividing the shear equally between the two flanges that cross the open panel, U3 – U5 and L3 – L5, the test load system failed to apply any axial force to the flanges. The view was taken that, because it was small, axial stress probably had little effect on flange behaviour compared with that of the bending stress. Any attempt to apply axial forces during the test posed considerable difficulty in ensuring that they remain truly axial while the flanges were rotating and deflecting significantly under load.

However, when the test load reached $5\,Q_T^*$, the relative deflection between the edge of the diaphragm and mid-panel was nominally four times that measured at $2\,Q_T^*$. As discussed subsequently in paragraph 8.10, the deflection attained under $2\,Q_T^*$ equates to long-term deflection. This indicated that the rotation at L4 and U4 and hence, the local stress concentrations developed in the long term under normal service conditions at the edge of the diaphragm at L3 and U3, would probably be magnified considerably by this test method for loads beyond $2\,Q_T^*$.

It was decided to resolve these issues by testing a true scale model of the full size wall. In the event, the truncated beam model was validated through tests on a one third scale model $3m$ long × $0.9m$ deep constructed with hardboard diaphragms.

8.8 SCALE MODEL

8.8.1 Model Design

To validate the truncated model's representation of a full 9*m* span wall beam, it is necessary that a scale model should

- reproduce proportionately the truncated model's deformation when subject to the scaled down equivalent of the test load that would create the equivalent long term deformation, that is $2\,Q_F^*$ and
- resist the model's equivalent of the full scale test load $5\,Q_F^*$.

To achieve this, the model must represent the full size wall beam scaled down to provide truly proportionate strength and stiffness and its behaviour must therefore reflect two sets of strength and deformability criteria, that of the diaphragm and that of the flanges crossing the open panel.

The model must also suit the available test facility, a major constraint being the 20*kN* maximum load that the Instrom load cell could apply to the top flange. The total prototype test load was thus limited to 40*kN*, half on the top flange and half on the lower flange, with the latter applied by another variable loading system, for this model, by a gravity system. Hence, only a one third, or smaller, scale model was feasible.

Particleboard around one third the 19*mm* thickness of the full size model diaphragm was not available. Therefore, a reduction to one third scale was only possible through the use of thinner material with mechanical properties similar to PB20 particleboard. This limited the choice to hardboard, which, as the literature indicated, exhibited similar mechanical behaviour. Values of the mechanical properties of hardboard drawn from various sources are given as 5%ile stresses and mean elastic constants in Table 8.4.

Table 8.4 Short Term Ultimate Mechanical Properties of Hardboard MPa

Source	S.G.	f_b'	f_c'	f_t'	f_{sp}'	E_b	E_c	E_t	G
AS/NZS1859-1997		40				4000			
CSR-1997	1.05	40				4000			
CSIRO-c1960*		42	32	32	20	4430		5050	
APPM-1962*		46		31		4600		5200	
Ramaker		42	32	31	20	5000	5800	4700	
Larsen	0.8+	50	25	30	19	3500	2700	2700	1350
Fidor	0.75	20	15		21	3000	2250	2000	1000
BSS 5669-1989		42	32	31	20			5000	2500

** Unpublished mean values.

The characteristic properties of hardboard manufactured by Australian Pulp and Pulp Mills (APPM) c1962 are given in Table 8.3. As referred to earlier, these properties, together with the CSIRO, (unpublished), duration-of-load factor of 0.24, made it possible for the writer to design and construct a range of beams with hardboard webs, some of which have seen nearly 40 years of service, refer Figure 8.1. The available diaphragm material for the scale model was hardboard manufactured in accordance with AS/NZS 1859-1997. Its modulus of elasticity of $4000\,MPa$ was nominally the same as that of Grade PB20 particleboard of $4100\,MPa$, therefore, a reasonable similarity between the elastic properties of particleboard and hardboard is seen to exist making a one third model possible.

To reproduce geometric and structural similarity with the full size beam, it is necessary to reproduce in the one third model's diaphragm the same shear stress and strain as in the full scale model. According to Buckingham's Π Theorem, within the elastic range

$$F_{1/3}L_{1/3}^{-2} = F_F L_F^{-2}$$

Where F is the applied force, L the length and the subscripts $1/3$ and F the one third scale and full scale models. It follows that

$$F_{1/3}/F_F = \left(L_{1/3}/L_F\right)^2 = 1/9$$

Therefore, when subject to $1/9$ of the full scale shear force, the stress in the one third model's diaphragm will match that in the full scale model, and its deformation will be $1/3$. A reduction in diaphragm thickness and beam depth to one third, respectively 6.4*mm* and 900*mm*, and the model's shear force to 1/9, satisfied this requirement. To match the scale effect in the flanges, it was necessary to proportion them to reproduce one third of the deformation of the truncated model when loaded within the elastic range. A pair of 125*mm* × 20*mm* MGP 12 members satisfied this requirement reasonably well. Similarly, the diaphragm/flange/stiffener connections were made with 2.0*mm* diameter nails that were spaced to produce one third of the nail joint stiffness of the truncated beam connections and hence, one third of the deformation.

Materials are as follows:	Flanges:		2 - 125×18 MGP12
	Stiffeners:	Edge	2 - 30×18 MGP12
		Middle	2 - 60×18 MGP12
	Diaphragm:		6.4 Bracing Grade Hardboard to AS1859-1995

The resulting one third scale model is shown in Figure 8.9.

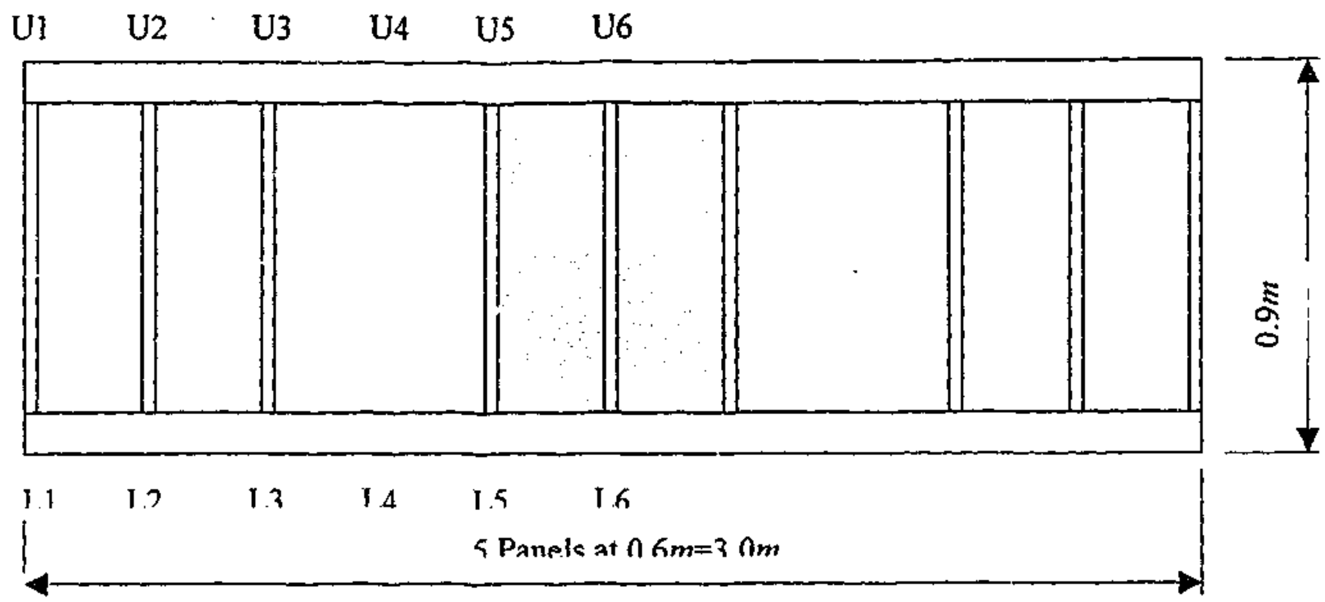


Figure 8.9 One Third Scale Model – Structural Details and Referencing Marking

8.8.2 Prototype Test Loading, $Q_{E1/3}$, for One Third Scale Model

The prototype test loading of $Q_{E1/3}=40kN$ for the one third scale model, which is one ninth of the full scale prototype test loading, is shown in Figure 8.10. To replicate the long-term deformation of the full size $9m$ beam that is estimated to occur at $2Q_F^*$, it is necessary to retain in the one third scale model the ratio $\frac{Q_{E1/3}}{Q^*_{1/3}}=5$. Values of $Q_{E1/3}$ are given in table 8.5.

Table 8.5 Prototype Test Load, $5Q_{E1/3}$ for One Third Scale model Beam

Component	$Q_{E1/3}(kN)$	$\frac{Q_{E1/3}}{Q^*_{1/3}}$
Diaphragms	40	5.0
Flanges	41	5.1
Nailed Connections	13	5.0

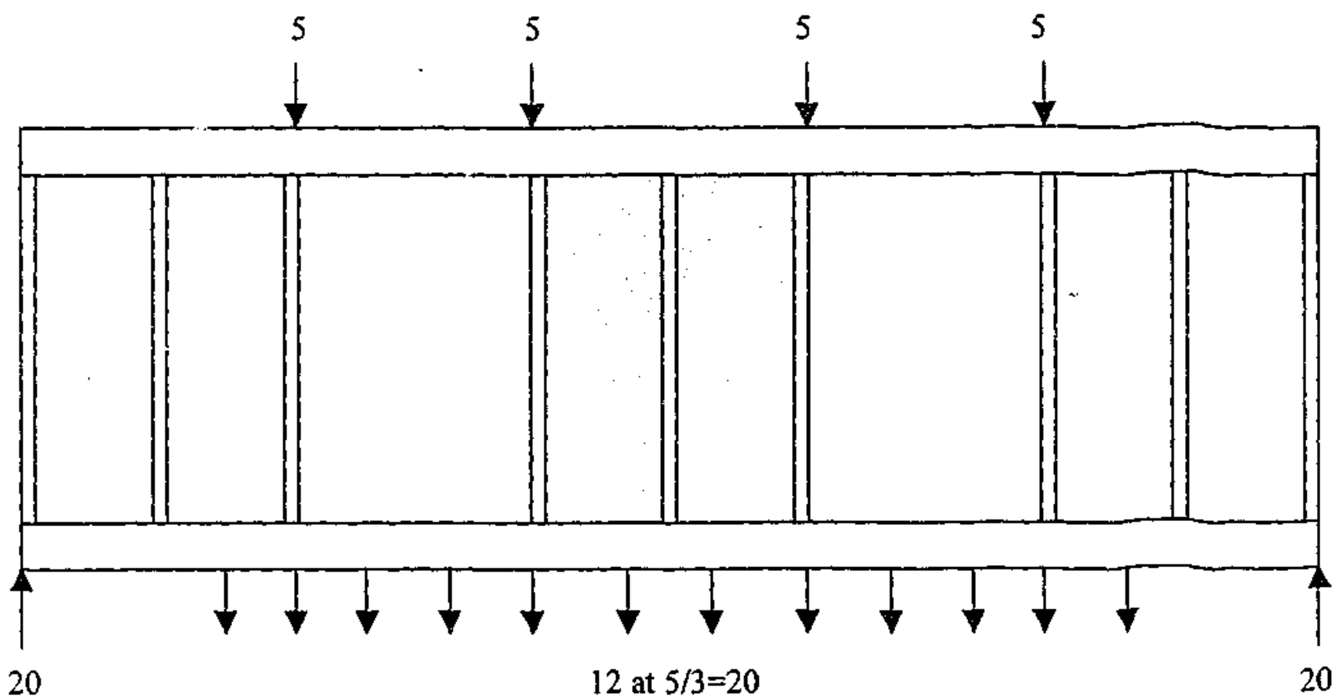


Figure 8.10 Loads and Reactions for One Third Scale Model Prototype Test Loading ($5Q^{*}_{1/3}$)

8.8.3 Test Method

As shown in Figure 8.11, the model is placed vertically in a loading frame that provides restraint against lateral buckling and the test load is applied through both top and bottom flanges. The beam is supported on a pin at one end and a roller at the other. The loading rate is arranged to reach $5Q^{*}_{1/3}$ in around 7 minutes, as for the truncated model. The load on the top flange is applied through an Instrom load cell and whiffle tree to the inner four panel points. On the bottom flange it was applied at 200*mm* centres by an adjustable dead weight system hydraulically controlled to apply the load gradually without impact. Thus, the load was applied to more closely represent the true load condition in the actual wall beam. To avoid local crushing, steel plates were inserted between model and load points. The model was bedded into its reactions before testing as described for the truncated model.

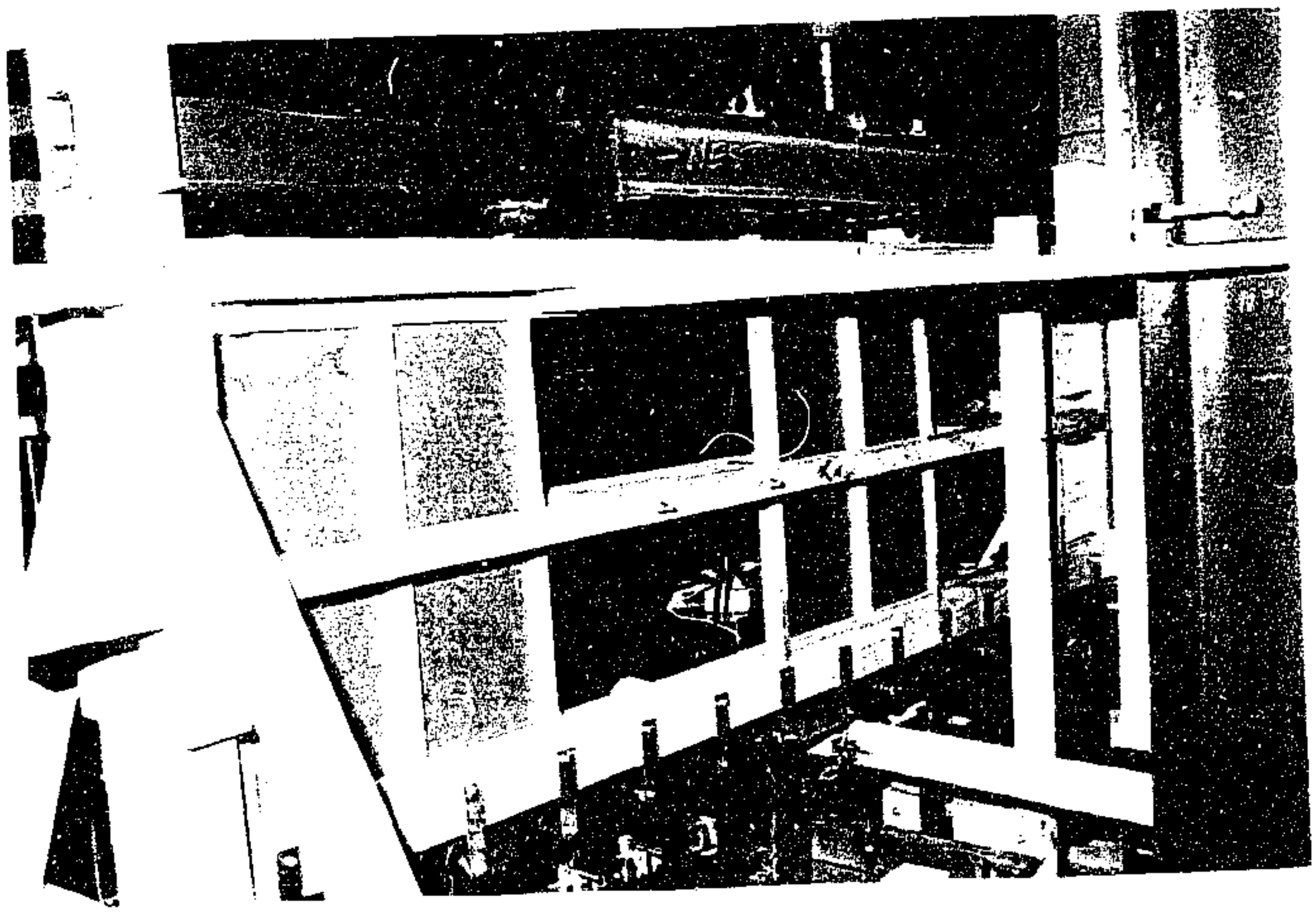
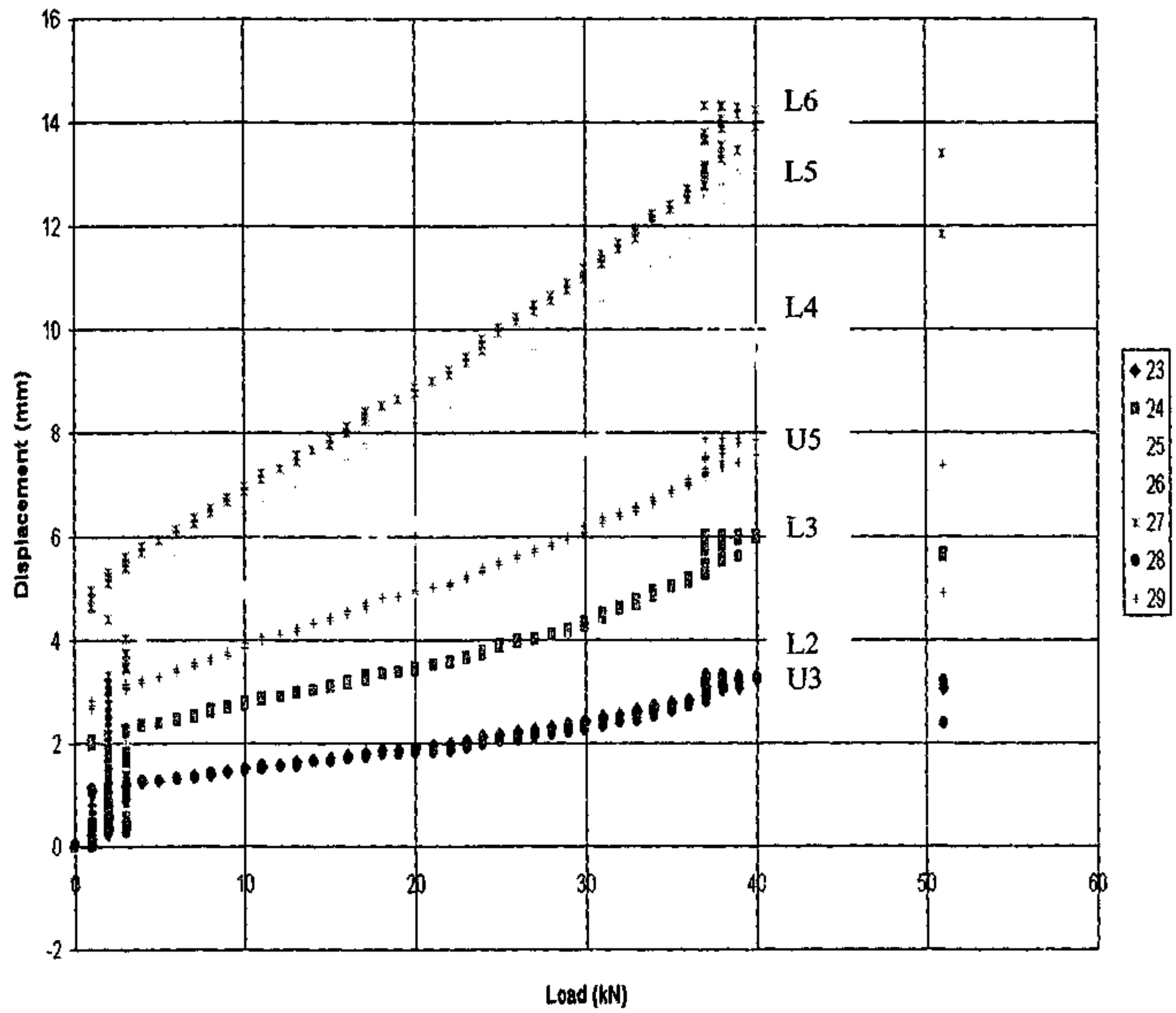


Figure 8.11 One Third Scale Model Test Rig Set-Up

8.8.4 Instrumentation

This model was instrumented to collect data similar to that on the truncated model. One end panel was strain gauged in six positions and the vertical movement of the bottom flange was recorded. All data was logged at two second intervals.

8.8.5 Deformation of One Third Scale Model



Legend : (Reference marking shown on Figure 8.9)

L2 to L6 Deflection of lower flange

U3 and U5 Deflection of upper flange

Figure 8.11 Deformation of One Third Scale Model

Table 8.6 Lower Flange Deflection at $2Q^{*}_{1/3}$=2.5 × Serviceability Design Load

Beam Model	Deflection (*mm*) at Panel Points and Mid-Panel Points					
	L1	L2	L3	L4	L5	L6
One Third	0	1.8	3.4	6.0	8.1	8.7

For practical purposes, the behaviour of the one third scale model was proportionally identical to that exhibited by the truncated model within the elastic range, refer Table 8.5. The deformation at $2Q^{*}_{1/3}=16kN$ represented the scaled down long-term deformation observed in the truncated model.

8.9 FINITE ELEMENT MODEL

8.9.1 Validation of Truncation of Experimental Model Wall Beam

To further validate the truncation of the full scale wall beam model, deformations predicted by a finite element model are compared with those determined by tests of both the truncated and one third scale models.

8.9.2 Description of Model

A finite element model of the one third scale model was assembled from plane stress and truss elements using the 1998, Strand 6.17, computer program. A 25*mm* plate mesh was sufficiently fine to provide the information sought. Face dimensions of the timber components were modified to suit 25*mm* multiples and adjustments to thickness to match the stiffness of their respective one third scale components were made accordingly.

Three plane stress plates subdivided into a 25*mm* square mesh were separately formed to represent the wall beam. One plate was fitted to the diaphragms, another to the flanges and web stiffeners, and the third to the steel plates that reinforce each outer diaphragm corner. The nails were represented as truss elements. The timber flange and stiffener element plates were laid over, and displaced 5*mm* horizontally and vertically from, the hardboard diaphragm element plates. To transfer shear stress between them, these plates were interconnected with truss elements at their common nodes. Plates representing the steel reinforcement were also displaced 5*mm* each way from the underlying plates to enable the formation of similar shear connections. The two truss elements were placed at 45° to the axis of each component and, thus, at right angles to each other. The three stress plates and interconnecting truss elements are delineated in Figure 8.13.

8.9.3 Structure Input Data

8.9.3.1 Flanges and Stiffeners

E= 12700*MPa,* Poisson ratio 0.3, thickness 36*mm.*

8.9.3.2 Diaphragms

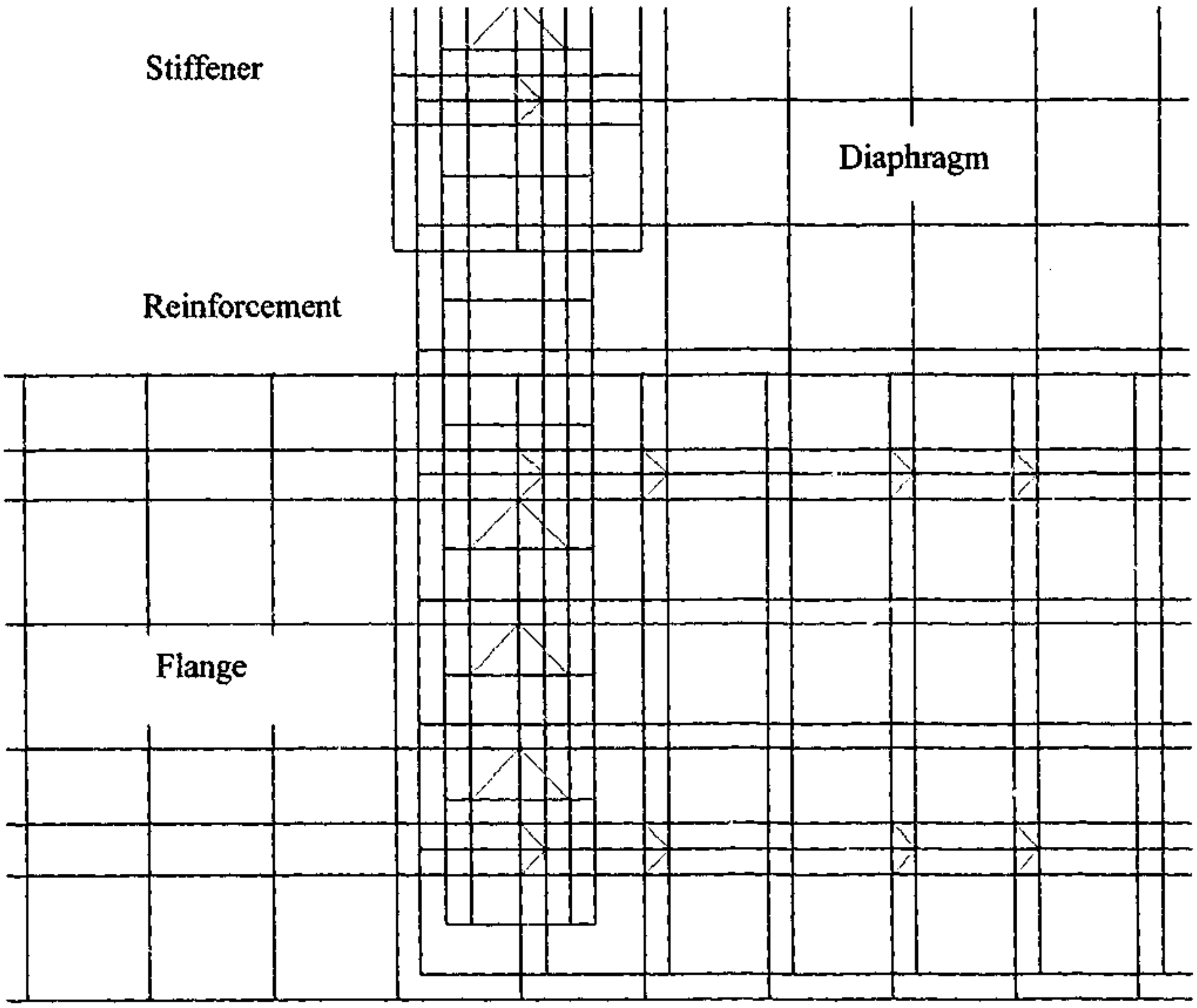
E= 4000*MPa,* Poisson ratio =0.3, thickness =6.4*mm*

8.9.3.3 Steel Reinforcement at Diaphragm Corners

E= 200000*MPa,* Poisson ratio =0.3, thickness =2.1*mm*

8.9.3.4 Nailed Connections

Nail Modulus=1440*N/mm,* A =3.14mm^2



Beam elements shown Red

Figure 8.12 Overlay Pattern and Nail Representation in Finite Element Model

8.9.4 Deformation of Finite Element Model

The deformed finite element model is shown in Figure 8.14 and predicted deflections along the lower flange are given in Table 8.7.

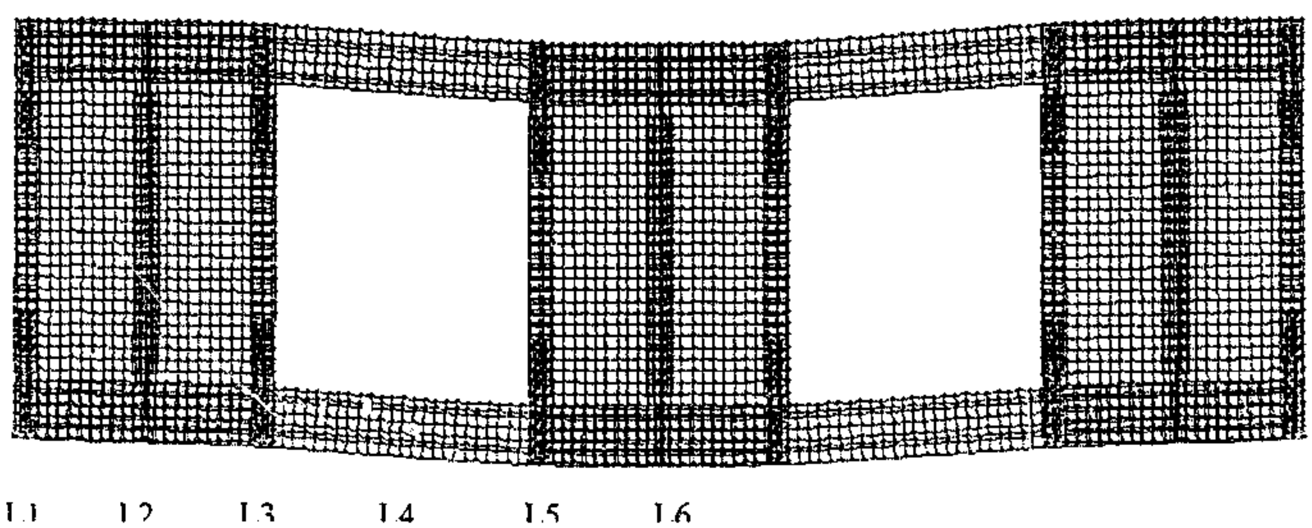


Figure 8.14 Deformation of Finite Element Model

Table 8.7 Lower Flange Deflection at $2Q^*_{1/3}$ in Finite Element Model

Beam Model	Deflection (*mm*) at Panel Points and Mid-Panel Points					
	L1	L2	L3	L4	L5	L6
Finite Element	0	1.6	3.2	6.2	8.9	9.5

8.10 COMPARATIVE DEFORMATIONS

8.10.1 Serviceability Loading

A comparison of the various model deformations under ultimate load is not relevant to an appreciation of the limiting deformation likely to be reached by the structure during long term service. Hence, this comparison is made under the load that is estimated to create the deformations ultimately developed in the structure that limit its serviceability.

The serviceability limit states load, S^*, determined in accordance with AS 1170.1-1989 is given by

$$S^* = G + \psi_l Q$$

where ψ_l is taken as 0.4 for floors generally, except storage or other special cases. The resulting serviceability limit states design load for beams spaced at 4*m* centres is thus

$$S^* = 4(1.4 + 0.4 \times 1.5) = 8kN/m$$

Taking the long-term deformation factor as 2.5, the deformation that occurs under a serviceability load of 20*kN/m*, therefore, provides the limiting conditions that the 9*m* span wall beam should satisfy, which load corresponds to twice the strength limit states design load, $2Q_F^*$. Similarly, for the truncated and one third scale models, when test loads reach twice their design loads, respectively $2Q_T^*$ and $2Q_{1/3}^*$, the deflections are measured.

8.10.2 Deflections

The resulting deflections for all models at $2Q^*$ are compared in Table 8.8.

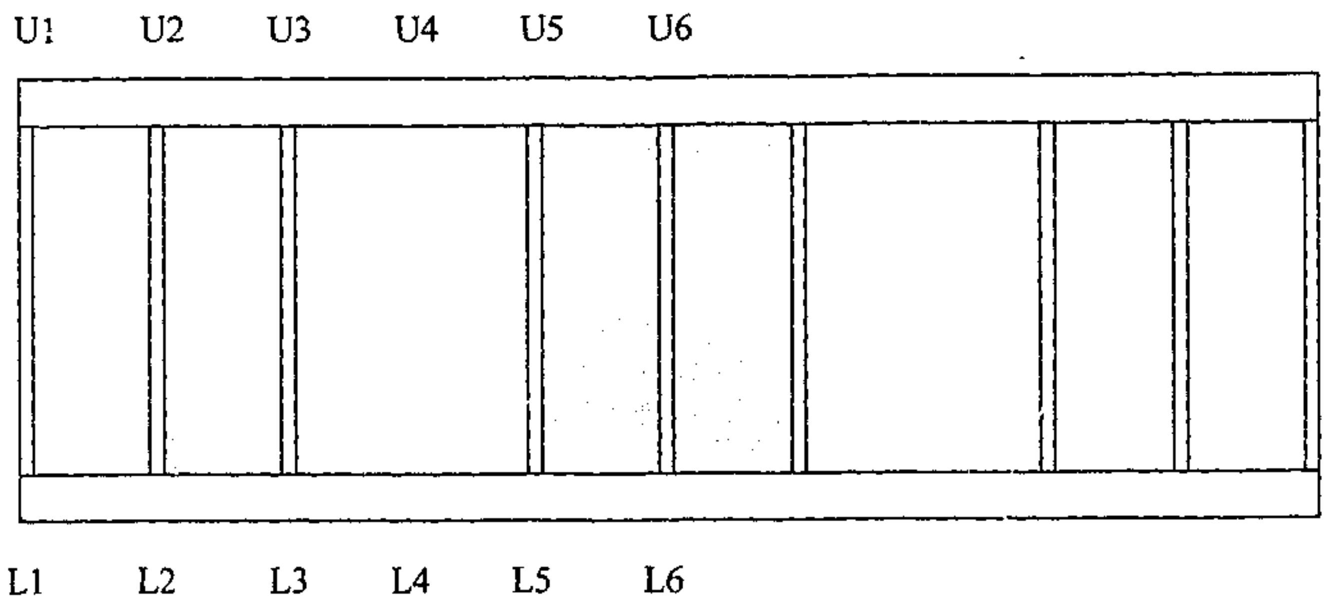


Figure 8.15 Reference Marking

Table 8.8 Lower Flange Deflection at 2.5 × Limit States Serviceability Design Load

Beam Model	Deflection (*mm*) at Panel Points and Mid-Panel Points					
	L1	L2	L3	L4	L5	L6
Truncated	0	-	11	19	-	-
One Third	0	1.8	3.4	6.0	8.1	8.7
Finite Element	0	1.6	3.2	6.2	8.9	9.5

8.11 DISCUSSION

Long-term deflections of the one third scale and finite element models agree reasonably well at all points. Scaling-up their deflections threefold to match the truncated model, the deflections of all three models are practically identical at C and D. It is concluded that a full size version of the experimental wall beam would deflect between 27*mm* and 30*mm* at F and therefore satisfy an acceptable deflection of span / 300. As referred to in paragraph 6.7.2, by treating 40% of floor live load as a dead load, the test load is some 20% greater than that when none is assumed to be permanent. The 30*mm* deflection is thus a more conservative prediction than that given by AS1710.1-1997 test loading for a prototype. This outcome supports the writer's view that the effects of transient live load are not relevant when designing timber structures and emphasises the importance of assessing more carefully, the true effects of so called live loads, that in fact, may be permanent.

The prototype test loads, Q_{EF}, Q_{ET}, $Q_{E\frac{1}{3}}$, are based on the strength duration-of-load factors of k_1=0.3 for particleboard and 0.48 for timber, the design and construction capacity factor for Grade C construction, ϕ_B=0.86, together with characteristic properties for particleboard which are some 25% lower than those given by AS/NZS 4063-1992. In the absence of published nail strengths specifically for particleboard, the experimental full size model is designed with nail strengths, estimated according to the empirical method described in AS 1720.1-1997, which are shown to be very conservative.

The structural design methodology complies with AS 1720.1-1997, except that in applying ϕ_B, the designer brings a measure of control into field supervision by penalising its absence. The resulting experimental model, truncated for test purposes, is fabricated in the field using normal carpentry trade skills, tools and equipment. To validate its truncation, a true one third scale wall beam model was designed and tested using procedures specified by AS 1720.1-1997 for prototype structures. The one third scale model replicated the truncated model's behaviour closely and confirmed that the truncation would satisfactorily represent a full size $9m$ span wall beam. Further verification was provided by the finite element model.

It is evident, in view of the conservative nature of the modification factors and the design and test methods employed, that this form of structure is stronger and stiffer than the test results suggest.

8.12 CONCLUSION

The stated aim of this study is to establish the fundamental basis and critical data for the structural use of particleboard and demonstrate that the material behaves satisfactorily when used as a primary element in a full-scale structure. The study demonstrates that, when so used in a structure designed in accordance with AS 1720.1-1997 and the findings of this study, particleboard of flooring grade quality conforming with AS 1859-1995 will behave both reliably and predictably in sheltered situations anywhere in Australia for at least 50 years.

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