CI/SfB

Date: February 1999

THE RESOURCE CENTRE FOR BUILDING EXCELLENCE

STUDY REPORT

No. 57 (1999)

Serviceability Limit State Criteria for New Zealand Buildings

Andrew B. King

The work reported here was funded by the Foundation for Research, Science and Technology and the Building Research Levy.

ISSN: 0113-3675

Preface

This report details serviceability criteria for New Zealand buildings. Its aim is to assist structural engineers and designers with decisions on deflection limits for various building components.

Acknowledgments

Support from the Foundation for Research, Science and Technology and the Building Research Levy is gratefully acknowledged.

Intended Audience

This publication is intended for structural engineers, architects, designers and building control authorities when they are attempting to satisfy the serviceability limit state provisions stipulated in the New Zealand Loadings Standard, NZS 4203.

SERVICEABILITY LIMIT STATE CRITERIA FOR NEW ZEALAND BUILDINGS

BRANZ Study Report SR 57 (1999) **Andrew B. King**

REFERENCE

King, A.B. 1999. Serviceability Limit State Criteria for New Zealand Buildings. BRANZ, Study Report SR 57, Judgeford.

KEY WORDS

Deflection, Vibration, Serviceability, Limit State Design.

CONTENTS

Page No.

Figures

Tables

Serviceability Limit State Criteria for New Zealand Buildings

Summary

Over recent years New Zealand has joined the international trend of prescribing structural performance and evaluating compliance by considering two limit states for building behaviour, namely behaviour at serviceability and ultimate limit states. As prescribed in the New Zealand Building Code each state needs to be considered and compliance ensured. Thus, while the traditional engineering design procedures concentrated on ensuring adequate capacity against collapse or instability, deformation considerations were of secondary consideration, whereas the current requirements demand equal importance be placed on ensuring that each state is satisfied. A major difficulty encountered by designers is that the performance criteria for serviceability limit state compliance have, at best, been loosely prescribed, and are either inappropriate for the given circumstances or totally absent.

This report ascribes serviceability criteria for different conditions and controls. The report primarily focuses on a table of suggested criteria for different materials and structural systems being used in various applications. It discusses the background and rationale for each criterion and includes typical values of movement-related parameters for materials commonly used as building elements. The final section of the report deals with designing against floor vibration problems. This includes discussion as to the nature of the problems, the various common forms these problems take and presents some of the more common methods of design assessment used to predict problem floors. Some worked examples are included to more fully describe these methods.

1. INTRODUCTION

Over recent years New Zealand has progressively revised and adopted structural design and material standards which are in limit states format. Building performance requirements are stipulated within the New Zealand Building Code (NZBC), which was published as Schedule 1 of the Regulations which were drafted to support the Building Act 1992 [New Zealand]. The structural performance objectives are stated in Part B1 of that document. These require building behaviour at both serviceability limit states (to preserve the building amenity) and ultimate limit states (where collapse or instability is to be avoided) to be assured. Although now formalised, little is new since both the stiffness and strength have long been the essence of structural design. However, now equal emphasis is placed on serviceability compliance, whereas previously deflection control was largely considered as somewhat of an afterthought. This shift in emphasis is well placed since the majority of building problems and failures relate to functionality issues rather than collapse.

This report addresses the issues relating to prescribing serviceability performance criteria for buildings and their components. It will primarily address the approach used to prepare the recommendation contained within the New Zealand Loadings Standard NZS 4203:1992 “Standard for General Structural Design and Design Loading for Buildings” (SNZ 1992). Some variations will be introduced, either on the basis of new information which has come to hand, or as corrections to information already published.

This guideline supersedes the BRANZ Study Report, SR 14, by Cooney & King [1988] in which serviceability criteria were previously described. It aims to assist structural engineers and designers to decide on suitable deflection limits for various applications by providing an explanation of how the published figures were determined, the ‘technical quality’ of the basis used and its applicability to the particular circumstance being considered. Section 7 provides guidelines for assessing acceptable levels of floor vibration. It forms a major part of this guideline and details specific design procedures relating to particular circumstances.

2. BACKGROUND

Prior to the 1960s the allowable design stresses assigned to most engineering materials were low and design methods were conservative. This resulted in highly redundant building forms, typically with comparatively short spans and relatively massive elements. Such buildings were generally very stiff to the extent that deflection problems were uncommon. There was little need to realistically ascertain the actual deformation of elements since these seldom controlled design or element sizes. By contrast, modern structures are generally lighter, possess less redundancy and are much more reactive to imposed loads. Modern structural design methods and material standards aim to realistically reflect the actual material properties and provide innovative designers with the tools to utilise the full potential of new materials. Material technology has also advanced with higher strength systems facilitating longer spanning elements, which are typically more susceptible to deformation anomalies. Designers assess the response of each element to the appropriate combination of realistic actions, often modelling these using sophisticated analytical and computer modelling techniques. The engineering rationale inherent within such an approach is complex. Several assumptions are required to assess that response, both to reflect the actual condition of the element in service and to ascertain the response of that element to the applied action.

This guideline identifies and discusses many of the assumptions that are made when assessing elemental deformation control. It provides more detailed background information to assist in assuring these assumptions are appropriate and provides guidance which should allow the sensitivity of such assumptions to be assessed with regard to the member, its physical properties or its in-service condition.

2.1 Structural Design To Limit States

The criteria for acceptable behaviour of buildings and their components requires that their response be within acceptable limits when subjected to stated actions. With limit states design, deformation control criteria usually apply to the serviceability limit state, and the intensity of the actions applied are intended to reflect conditions which are expected to occur with a 5% probability of excedence in any year of service. Conversely stability criteria of both adequate strength and collapse avoidance apply to the ultimate limit state, where the actions applied reflect conditions which are expected to occur with a probability of excedence of between 5 and 10% over the intended design life of the building, typically considered as being 50 years. The actions imposed are generally combinations of loads, and system compliance requires that all combinations which can reasonably be expected to occur will be considered. Although the true combination of actions may be large, designers will generally be able to disregard many as trivial and concentrate only on those which combine to impose the most severe actions on the structure. Other, less critical combinations, can be ignored.

Buildings that exceed appropriate serviceability limit states generally manifest this transition with excessive cracking, unsightly deformations, accelerated deterioration or vibration problems.

2.2 The New Zealand Legal Framework

The New Zealand Building Code (NZBC), which is called as Schedule 1 of the regulations relating to the New Zealand Building Act, passed into law in July 1992. After a transition period, it became the sole basis for regulated building compliance and thus occupancy approval for New Zealand buildings in January 1993. The NZBC is presented in 37 parts, each of which prescribes the performance expected by the complete building with regard to a specific attribute (ie stability, fire, weather resistance, energy efficiency, etc.). Building compliance with the Code requires compliance of the building system with each part of the code. The presentation style of each part is identical. The overall *objective* of the part is stated (what the goal sought by the provisions). The *functional requirements* to be met in achieving the stated objective, and the *performance requirements* needed to satisfy the functionality provisions follow and relate back to that *objective*. Although the Code is performance based, there are few examples where the performance is quantified within these mandatory sections of the Code. Rather a set of Approved Documents have been published as an adjunct to the Code. The Approved Documents contain the means by which the stated *objectives* can be satisfied, typically detailing acceptable Verification Methods (basis for design) and Acceptable Solutions (prescriptive deemed-to-comply solutions). The underlying philosophy throughout the code is to ensure occupants and neighbours enjoy an adequate level of health and safety and that the amenity value of the building is maintained. Beyond these, 'market forces' are expected to drive building quality and performance.

The structural requirements are contained within Part B1 'Structure', and are presented in conjunction with B2 Durability as Part B of the Code. The *objective* of this provision (Clause B1.1) is to safeguard people from injury caused by structural failure and from loss of amenity caused by structural behaviour, and to protect other property from physical damage due to structural failure. The *functional requirements* (Clause B1.2) stipulate that the building and its components shall withstand the combination of loads to which they are likely to be subjected during their life. The *performance* provisions (Clause B1.3) specify that buildings (and building elements) are to have a low probability of collapsing or becoming unstable throughout their life (Clause B1.3.1), and that they have a low probability of losing their amenity value (usefulness) through deformation, vibration, degradation or other changes to their physical characteristics (Clause B1.3.2). Thus it is compliance with the loss of amenity provisions of Part B1 of the NZBC that demands that the building remains serviceable.

3. SERVICEABILITY LIMIT STATE CRITERIA TABLES (REFER TO TABLES 1 & 1A)

3.1 General

An acceptable evaluation method specified within the Approved Documents of the NZBC is the New Zealand Loadings Standard, NZS 4203 [SNZ 1992]. The specific provisions that relate to the Serviceability Limit State of buildings are contained with Clause 2.5.2 of that Standard, which states:

"2.5.2.1 For the serviceability limit state, the deflections of the structure shall not be such as to result in damage causing loss of function of the structure or its parts

2.5.2.2 Where departure from linear elastic behaviour is assumed in analysis, the effect of the resulting deformation on serviceability shall be considered"

The load combinations applicable for serviceability limit state considerations are published in Clause 2.4.2 and are included, along with background discussion in Section 5 below.

Whilst the clause provisions of the loading standard stipulate the intent of the serviceability provisions, the actual serviceability criteria are published in the commentary provisions to Clause 2.4. These were mostly extracted from Cooney and King [1987] and re-formatted and published as "Suggested Serviceability Limits for

Deflection" in Table C2.4.1 in the commentary to the Loadings Standard [NZS 4203, SNZ 1992]. The combination of applied design action and limits of acceptable response combine to provide quantified serviceability criteria which designers can use to demonstrate code compliance using rational engineering design. They therefore play a pivotal role during the design process.

This publication provides an insight to the basis for the values published, some explanation regarding how they were derived and clarification to assist in their correct interpretation. The designer is expected to be able use the criteria published and apply engineering judgement when assessing and applying the limits specified to confirm that they are appropriate for the use to which they are being applied. This concept of providing guidelines for designers rather than definitive rules was the intent of the Loadings Standard review committee. The inclusion of the commentary of NZS 4203 as part of the Verification Method within Clause B1 of the Approved Documents to the NZBC has largely negated this intent. While Approving Agencies continue to expect compliance with the requirements stipulated in Table C2.4.1 of NZS 4203, it is anticipated that with this publication of the technical basis for the criteria designers will be able to develop appropriate rational arguments to justify departures from the criteria specified in some cases.

Table 1 of this publication presents an expanded set of serviceability limit state criteria which are applicable for New Zealand buildings. Many are similar to those published in Table C2.4.1 of the commentary to NZS 4203 [SNZ 1992]. Some criteria have changed however, usually as a result of additional work which has been undertaken during the interim period. The criteria have been expanded to include new criteria where these have been established. All such changes have been marked in Table 1 with an asterisk for easy identification. The presentation style has also been retained and background to that style is expanded upon in Section 4 below. An alternative, more user friendly form of presentation is presented in Table 1A. In both cases the tables continue to use 'Suggested Serviceability Limit State' rather than making any firm recommendation. This is intended to be recognition that the boundaries between 'acceptable' and 'unacceptable' behaviour, particularly with respect to serviceability, are poorly defined and to encourage designers to consider each case on its merits, using the values stipulated as a guide only.

A reference identification column has been added to enable cross reference between Table 1 and Table 1A. The first letter of the reference identification relates to the serviceability phenomenon controlled which forms the primary focus of Table 1. The criteria themselves are unchanged between the two tables.

Table 1: Suggested serviceability limit state acceptance criteria (based on NZS 4203:1992)

Control Phenomenon	Ref id	Element Being Assessed	Serviceability Aspect Under Consideration	Imposed Design Action	Limit of Acceptable Response
Sensory: Seen	S1	Metal roof claddings*	Indentation	Residual after Qb = 1 kN	Span/600 & <0.5 mm
	S2	Roof members (trusses, rafters, purlins, etc.)	Sag	G & Ψ_LQ	Span/300
	S3	Ceilings with matt or gloss paint finish	Ripple	G	Span/500
	S4	Ceilings with textured finish	Ripple	G	Span/300
	S5	Suspended ceilings (rails with inset tiles)	Ripple	G	Span/360
	S6	Ceiling support framing	Sag	G	Span/360
	S7	Glazing systems	Bowing	Ws	Span/400
	S8	Columns	Side sway	Ws	Height/500
	S9	Flooring*	Ripple	G & Ψ_LQ	Span/300
	S10	Floor joists/beams*	Sag	G & Ψ_LQ	Span/300*
	S11	Beams where line-of-sight is along invert	Sag	G & Ψ_LQ	Span/500
	S12	Beams where line-of-sight is across soffit	Sag	G & Ψ_LQ	Span/250
Sensory: Felt	S13	Walls (face loaded)	Discernible movement	Ws	Height/300
	S14	Walls (impact)	Neighbours notice	Qb = 1.2 kN*	Height/200 & <12 mm
	S15	Floors (lightly damped eg solid construction)	Vibration	Qb = 1.0 kN	<1 mm*
	S16	Floors (highly damped eg joist & deck)	Vibration	Qb = 1.0 kN	<1.5 mm*
	S17	Floors - side sway	Sway	Ws	<0.01 g
Functionality	F1	Flat roof systems	Drainage	G & Ψ_LQ	Span/400
	F2	Flat roof systems	Ponding & melting snow	G & Ss	Span/500
	F3*	Flooring* (Differential deflection across supports)*	Tilt	G & Ψ_LQ	0.002 Radian
	F4*	Flooring over specialty floors*	Ripple	Qb = 1.0 kN	Span/800 & < 0.5 mm
	F5	Specialty floor systems	Noticeable sag	G & Ψ_SQ	Span/600
	F6	Lintel beams (vertical sag)	Doors/windows jamming Metal facia buckling	G & Ψ_SQ	Span/300 * & <10 mm
Protection of non-structural elements	D1	Concrete or ceramic roof claddings*	Cracking	Qb = 1.0 kN	Span/400
	D2*	Metal roof claddings*	Seam de-coupling	G & Ψ_SQ	Span/120
	D3	Ceilings with plaster finish (normal to plane)	Cracking	G & Ψ_SQ	Span/200
	D4	Walls with brittle cladding (normal to plane)	Cracking	Ws	Height/500
	D5	Fixed glazing systems (in plane)	Glass damage	Ws; Es	Height/300 & < twice glass clearance
	D6	Windows, facades, curtain walls (normal to plane)	Facade damage	Ws	Span/250
	D7	Masonry walls (in plane)	Noticeable cracking*	Ws; Es	Height/600
	D8	Masonry walls (normal to plane)	Noticeable cracking	Ws; Es	Height/400
	D9	Plaster/gypsum walls (in plane)	Lining damage*	Ws; Es	Height/300*
	D10	Plaster/gypsum walls (normal to plane)	Lining damage*	Ws; Es	Height/200
	D11	Plaster/gypsum walls (Equivalent soft body impact)*	Lining damage*	Qb = 0.7 kN	Height/200
	D12	Movable partitions (equiv. Sb impact)	System damage	Qb = 0.7 kN	Height/160
	D13	Walls with other linings (normal to plane)	Lining damage	Ws; Es	Height/300
	D14	Portal frames (frame racking action)	Roofing damage	Ws; Es	Spacing/200
	D15	Floors/beams - supporting masonry walls*	Wall cracking	G & Ψ_SQ	Span/500
	D16	Floors/beams - supporting plaster lined walls*	Cracks in lining	G & Ψ_SQ	Span/300

Table 1A: Suggested serviceability limit state criteria (alternative presentation)

Element Being Assessed	Ref id	Serviceability Aspect Under Consideration	Imposed Design Action	Limit of Acceptable Response
Roof Claddings				
Metal roof claddings*	S1	Indentation	Residual after Qb = 1 kN	Span/600 & <0.5 mm
	D2*	Seam de-coupling	G & Ψ_sQ	Span/120
Concrete or ceramic roof claddings*	D1	Cracking	G & Ψ_sQ	Span/400
Roof Supporting Elements				
Roof members (Trusses, rafters, purlins, etc.)	S2	Sag	G & Ψ_sQ	Span/300
Roof elements supporting brittle claddings	D1	Cracking	G & Ψ_sQ	Span/400
Ceilings & Ceiling Supports				
Ceilings with matt or gloss paint finish	S3	Ripple	G	Span/500
Ceilings with textured finish	S4	Ripple	G	Span/300
Suspended ceilings (rails with inset tiles)	S5	Ripple	G	Span/360
Ceiling support framing	S6	Sag	G	Span/360
Ceilings with plaster finish (normal to plane)	D3	Cracking	G & Ψ_sQ	Span/200
Wall Elements				
Columns	S8	Side sway	Ws	Height/500
Portal frames (frame racking action)	D14	Roofing damage	Ws; Es	Spacing/200
Lintel beams (vertical sag)	F6	Doors/windows jamming Metal facia buckling	G & Ψ_sQ	Span/300 * & <10 mm
Walls (face loaded)	S13	Discernible movement	Ws	Height/300
	S14	Neighbours notice	Qb = 1.2 kN*	Height/200 & <12 mm*
Walls with brittle cladding (normal to plane)	D4	Cracking	Ws	Height/500
Masonry walls (in plane)	D7	Noticeable cracking*	Ws; Es	Height/600
	D8	Noticeable Cracking	Ws; Es	Height/400
Plaster/gypsum walls (in plane)	D9	Lining damage*	Ws; Es	Height/300*
	D10	Lining damage*	Ws; Es	Height/200
	D11	Lining damage*	Qb = 0.7 kN	Height/200
Movable partitions (equiv. SB impact)	D12	System damage	Qb = 0.7 kN	Height/160
Walls with other linings (normal to plane)	D13	Lining damage	Ws; Es	Height/300
Glazing systems	S7	Bowing	Ws	Span/400
Windows, facades, curtain walls (normal to plane)	D6	Facade damage	Ws	Span/250
Fixed glazing systems (in plane)	D5	Glass damage	Ws; Es	Height/300 & < twice glass clearance
Floors & Floor Supports				
Beams where line-of-sight is along invert	S11	Sag	G & Ψ_LQ	Span/500
Beams where line-of-sight is across soffit	S12	Sag	G & Ψ_LQ	Span/250
Flooring*	S9	Ripple	G & Ψ_LQ	Span/300
Floor joists/beams*	S10	Sag	G & Ψ_LQ	Span/300*
Floors (lightly damped eg solid construction)	S15	Vibration	Qb = 1.0 kN	<1 mm*
Floors (highly damped eg joist & deck)	S16	Vibration	Qb = 1.0 kN	<1.5 mm*
Floors - side sway	S17	Sway	Ws	<0.01 g
Flooring* (Differential deflection across supports)*	F3*	Tilt	G & Ψ_sQ	0.002 Radian
Flooring over specialty floors*	F4*	Ripple	G & Ψ_sQ	Span/800 & < 0.5 mm
Specialty floor systems	F5	Noticeable sag	G & Ψ_sQ	Span/600
Floors/beams - supporting masonry walls*	D15	Wall cracking	G & Ψ_sQ	Span/500
Floors/beams - supporting plaster lined walls*	D16	Cracks in lining	G & Ψ_sQ	Span/300

Notation

G= Self weight (Dead Load)

Q = Design Occupancy (Live Load)

Qb = Basic Live Load (to be applied unfactored)

Ψ_s = Short-term Live Load Duration Factor

Ψ_L = Long-term Live Load Duration Factor

Ws = Wind Forces (serviceability)

Es = Earthquake Actions (serviceability)

* indicates change of criteria from that published in Table C2.4.1 of NZS 4203

Table 1 follows the general layout of Table C2.4.1 of NZS 4203 [SNZ 1992] where the primary focus or entry point is the serviceability phenomenon being controlled (i.e. sensory, functionality or secondary damage). The specific element being assessed and the serviceability aspect of concern are followed by the applied actions or combination of actions and the limit for acceptable response. The background for the development of each criteria is given in Section 3.2. Table 1A is simply a rearrangement of Table 1 wherein the focus changes to being the element under consideration rather than the serviceability phenomenon. This evolved from comments by designers that during the design process, they tend to work from an elemental approach and would prefer this alternate layout. A simple cross reference identification label is provided to allow easy cross referencing where the first letter of the index relates to the control phenomenon. Where more than one criterion applies to an element each will need to be checked with the most severe governing. In the case of floor liveliness controls, a simple stiffness of maximum deflection under a static point load action is suggested as a trigger value before requiring a more detailed study (refer to Section 7). The results of any such study are themselves indicators of an anticipated lively floor problem and will override the excedence of the trigger value. The symbols used in Table 1 and Table 1A are fully described in Section 5. Criteria marked with an asterisk are different from those previously published in Table C2.4.1 in NZS 4203 [SNZ 1992] and are marked for easy identification.

3.2 Background Considerations to the Criteria

In deriving the criteria, the following principles were considered:

- Serviceability criteria are often subjective and defy strict limits of acceptability. The criteria stipulated are those which will have a low probability of causing concern to people using the building. These criteria have been derived to reflect normal occupancy and use. The list published is neither exhaustive nor definitive. It is presented as a guide only and should be adjusted either more liberally (for temporary or industrial buildings) or more rigidly (for special purpose buildings such as gymnasia, student hostels, etc).

- The designer is encouraged to understand which serviceability controls apply for each given situation. Table 1 presents the criteria sorted according to the control phenomenon (i.e. why limit the deformation) which was expected to provide a ready means of eliminating unnecessary options. Table 1A presents the same criteria but sorted according to the element being assessed which is more aligned to design office practice. The criteria are cross referenced by the reference identification key, the first letter of which provides some indication of the control phenomenon.

- When more than one serviceability criterion is applicable, each is to be satisfied, with the most demanding effectively controlling the design of that element. Care should be given to ensure that transient effects, such as creep and thermal effects, are consistent for the various regimes considered.

- Individual elements are usually linked to create structural systems. When subjected to load, it is the resistance of the complete system which should be considered. The apportionment of load between elements can be established from the degree of interaction between elements and their relative rigidity.

- The degree of end fixity present will also significantly affect the deformation of structural systems and needs to be carefully assessed. Normally greater end restraint, while reducing deformation, will attract more load and thus place greater demand on the strength capacity of the element. A realistic assessment of boundary condition rigidity needs to be made and applied consistently over both limit states.

- Traditional deflection criteria have been changed only when there was a sound technical or practical reason supporting such a change. Apparent changes in criteria have often been brought about by changes in the load intensity as the serviceability loads have become more clearly defined. The lack of hard data upon which to base rational criteria remains a problem. Work is continuing within this area and further changes are likely as the results from this work become available.

- A stated limit should represent a point where there is a transition from an acceptable to an unacceptable condition. Thus response beyond the stated limits should be expected to result in the onset of an unsatisfactory response (cracking, complaints, etc). Functionality related criteria and sensory control are both highly judgemental and are even more difficult to accurately quantify.

3.3 The Criteria

Sensory Control

S1 Aim: To prevent metal roof cladding from residual deformation after people have walked over the roof (either during the erection of the roof or during ongoing maintenance). This control aims to avoid unsightly depressions (refer also to D2 below). The action should be considered as being applied through a 100 mm diameter rubber pad, and the residual deformation assessed at the point of application relative to the roof plane at a 300 mm radius from that point.

S2 Aim: To prevent excessive deflections in roof supporting members thereby avoiding unsightly sagging of the roof sheathing. The stated criteria apply to roof systems which are reasonable and regular (eg profiled metal claddings) and could be relaxed for highly textured roof profiles such as concrete or ceramic tiles. Particular care should be taken at ridge lines where relative apex movement of adjacent trusses or sag of ridge beams will be more readily seen. Increasing the rigidity of truss interconnecting members will act as a load sharing mechanism, which would assist with this problem. Long-term shrinkage effects need to be considered.

S3 Aim: To prevent unsightly undulations across paint-finished ceilings. Reference to 'ripple' as the serviceability aspect under consideration is related to the wave-like effect of planar surfaces at they sag between points of support. In such cases the control span is the distance between those points of support (i.e. joist, ceiling batten or truss spacings). Since the phenomenon being controlled is visual perception, the acceptance limit may be relaxed if the ceiling is higher (i.e. well above the line-of-sight), or the room shape is such that reflected light is not easily seen over the ceiling plane. Conversely the acceptance limit will need to be tightened (to perhaps twice that suggested) in cases where the angle of incident light is very low (e.g. a skillion roof with clearlight windows) where the sunlight passes close to parallel to the ceiling plane.

S4 Aim: To prevent ceilings with textured finishes from unsightly undulations between points of support. As with S3 above, but with the textured ceiling finish dispersing the light from the surface and thus avoiding the reflective component of the appearance.

S5 Aim: To prevent panelised suspended ceilings from visually unacceptable undulations. The sight lines for such systems are generally the in-set tile surface (rather than the support rails). Such surfaces are often textured and generally broken by the rail grid pattern. Planar variations are more difficult to see and greater tolerances allowed before ripples become apparent. Where suspended ceiling framing is used to support flat board ceilings, then the criteria in S3 and S4 would apply.

S6 Aim: To prevent discernible sag in elements which support ceilings. This control applies to support elements (ceiling battens, purlins, bottom chords of trusses), the displacement of which translates directly to the ceiling itself. Although the element being controlled is hidden, the criterion specified reflects the displacement at which sag in the ceiling plane itself becomes noticeable when used from below. More stringent limits will apply when the ceiling abuts, but is not supported by, fixed vertical elements such as walls or partitions. An example of this latter issue would be the mid-span edge zone of a ceiling which is supported on one side by a simply supported flexural member and by a rigid wall or partition on the other. A large tilt will occur over the ceiling plane over this zone.

S7 Aim: To prevent out-of-plane bowing of vertical glazing systems (eg windows or facades). The criterion stipulated was derived by transcribing the traditional working stress limit (span/180 with a nett pressure differential of 1.2) to serviceability limit wind speeds with nett pressure differentials of 1.0 (ie windward wall external pressure of +0.7 and internal suction of -0.3 with two opposite walls being considered equally permeable.

S8 Aim: To prevent discernible column side sway. This criterion stipulates lateral action resulting from serviceability intensity wind speeds. It was derived from the previous control criterion (H/360) modified by a factor to reflect differences between working stress and serviceability wind speeds. Note that visual control criteria do not apply for serviceability earthquake loads. Observers are likely to be severely distracted even during a moderate earthquake which makes visual sensory criteria an inappropriate control.

S9 Aim: To prevent floor sheathing from discernible ripple or indentation of the flooring between lines of support. The span referred to within the criterion is the spacing between support joists. The criterion suggested applies to matt finish floors with a moderate intensity of light reflection from the floor surface. A 50%

more stringent control would be appropriate for highly reflective surfaces such as vinyls or gloss varnish. A 30% reduction in the control would be appropriate when carpet or cushioned floor coverings are used.

S10 Aim: To prevent floor joists and/or floor support beams from noticeable sag. This criterion controls the overall deflection of a flooring system where sag is likely to be seen. The 'span' referred to within the criterion may either be the simple along-joist clear span or the diagonal dimension between fixed support points (of particular significance when compound deflections are significant, such as when flexural members are supported on flexible secondary supporting members). Since floor joists often provide direct support for the ceiling below, ceiling criteria should also be checked.

S11 Aim: To prevent noticeable beam or joist sag where there is line-of-sight along the beam invert. Such conditions commonly occur in stair shafts or on steeply sloping sites. The beam invert provides a clear reference plane against which deflections can be more readily gauged. Car-park buildings, which often have reduced head clearance because of their split level form, and where the observer can be expected to move to adjacent levels along the ramps, are one such instance where this level of control has been found to be necessary.

S12 Aim: To prevent noticeable beam sag where there is a line-of-site across the beam invert. In these cases there is seldom a ready reference plane and the acceptable limit has been relaxed.

S13 Aim: To prevent wall systems from becoming so flexible that surface movements are sufficient to cause concern to building occupants. Precursors to such alarm often involves items rattling on shelves supported directly off the wall. Although external walls are the most susceptible to this action (because the wind pressure differential is greatest), internal partitions are also expected to resist differential wind pressure of approximately half those of applied to the building envelope.

S14 Aim: To prevent wall systems from being so flexible that movement is noticed when the wall is hit by accidental impact loads (e.g. people falling against the panel or doors slamming). The criterion stipulated is the lateral static load that is equivalent to a 200J impact response from a soft-body impactor. The 200J level is appropriate for normal office or residential use. More vigorous activity areas (gymnasia, sports halls etc.) may require greater impact resistance levels. Similarly industrial and some commercial areas require adequate indentation resistance. This can be measured by dropping a metal ball onto the surface under consideration or by measuring the force required to press a ball into the surface. Such information is generally available from system suppliers. The 12 mm deflection limit aims to avoid alarm since impacts are usually sudden and unexpected events, which often occur in an otherwise tranquil environment. Walls which support shelving or from which paintings or mirrors are hung, may need to be much stiffer (2 times) if movement is not to be noticed.

S15 Aim: To identify potentially lively, lightly damped floors as a means of triggering more detailed dynamic assessment. The use of a simple trigger was considered appropriate to avoid the need for such a detailed investigation in all cases. Floor systems supporting open plan rooms, particularly where there is little cross connection between joists, are expected to be lightly damped and fall within this trigger value. The acceptable performance of the floor system is determined through the more detailed dynamic analysis prescribed in Section 7 below, which will take precedence over the trigger value.

S16 Aim: To identify potentially lively more heavily damped floors as a means of triggering more detailed dynamic assessment. More detailed methods of evaluating the acceptability of floor systems are prescribed in Section 7 below. The results from more detailed studies should be considered more accurate than the acceptance guide and should be given priority.

S17 Aim: To prevent side sway of buildings (floors) when subjected to wind. The acceptability limits are from a study undertaken by Cenek & Wood (1990) which quantified a simple trigger value which identified 'wind sensitive' buildings (which require dynamic wind design techniques to be applied). It drew on wind induced lateral vibration studies undertaken by Melbourne & Cheung (1988) and by Galambos (1973).

Functionality Control

F1 Aim: To ensure adequate drainage of flat roof systems is maintained. This provision is primarily to avoid the development of sag-induced reverse falls from which ponding and backfall water build-up can result.

F2 Aim: To ensure ponding is avoided on flat roofs, balconies and exposed patios. Similar to F1, but with water or snow buildup being the initiating mechanism rather than imposed live load. While normally the aspect controlled ensures adequate drainage to preclude puddles etc, in rare extremes, ponding problems of this type can result in structural instability as the sag induces ponding which promotes greater sag through to failure.

F3 Aim: To prevent floor sheathing from discernible tilting between support lines. Where deflections may be localised (eg within the mid-region of a joisted floor when the adjacent support is a non-flexural member such as a wall – refer S6 above) then the local tilt (being the relative settlement of one support relative to the other) should be less than the acceptable response limit. This affects the ability to locate furniture or built-in fittings anywhere over the floor without it becoming inoperative. Thus tall items of furniture can rock or tilt, and cupboard doors swing open when these limits are exceeded.

F4 Aim: To prevent floor sheathing from discernible ripple between support lines. The specialty nature of these floors is envisaged to involve wheeled trolleys etc which 'rock' as they are rolled along the floor. Abrupt disruptions in surface flatness may also be a problem in these cases with 'steps' as small as 0.5 mm being sufficient to disrupt the smooth travel of firm wheeled trolleys.

F5 Aim: To prevent specialist floor areas from sag related problems when subjected to short term live loads. Floors such as those supporting precision optical equipment (photographic or Xray) or those where particular levels of flatness are required (eg bowling alleys) would be within this category. Designers are encouraged to seek precise specifications for floor alignment from the equipment manufacturers.

F6 Aim: To prevent lintel support beam sag effects which may render the adjacent non-loadbearing element inoperative. Window or door jamming as a result of lintel sag is the outcome for which protection is intended. The 12 mm maximum is approximately the clearance between the lintel and the frame head.

Protection of Non-structural Elements

D1 Aim: To prevent damage to brittle roof cladding elements such as concrete or ceramic roof tiles or fibre-cement shingles when subjected to maintenance loads. The criterion relates to the span of the individual tile (rather than the supporting tile batten) since it is assumed that movement is possible between individual tile elements themselves. The applied action is intended to represent a concentrated (footprint) load of 1kN and is applied over a 100 mm diameter rubber bearing pad. If the system includes cement-based mortar between elements, the batten span should be used as the basis for control.

D2 Aim: To prevent metal roof cladding systems decoupling, particularly along the seams of elements. The residual indentation check (S1 above) should also be considered with regard to water ponding on flat roof pitches as ponded water is likely to adversely affect the durability of most sheet metal products.

D3 Aim: To prevent cracking within plasterboard ceiling systems. Although the imposed action includes the short term live load, this only needs to be imposed on the bottom chord of supporting trusses or on the ceiling runner, not the ceiling itself.

D4 Aim: To prevent brittle wall cladding systems (such as ceramic tiles) against the onset of cracking or loss of bond because of the development of wall curvature under wind face loads. The height reference is the distance between points of support of the substrate.

D5 Aim: To prevent fixed glazed systems against in-plane racking induced by serviceability wind or earthquake actions. Fixed glazing is more directly attached to the structural frame and has less support flexibility to accommodate interstorey drifts. Hence twice the edge clearance between frame and glass is an appropriate control. Allowing for glazing pane rotation within the frame is acceptable. Contact between the glass and frame across the diagonal should be avoided and is thus a reasonable control.

D6 Aim: To prevent glass, glazing elements, windows and curtain wall systems against loads normal to the plane of the panel. Such loads would usually result from wind pressure applied directly to the face of the element. Decoupling, loss of system integrity and potential weather tightness problems are the aspects being addressed. The ‘span’ referenced in the acceptable limit refers to the clear span of the supporting frames, mullions or transoms. Glass or windows falling from the frame are considered to constitute a similar hazard to building collapse. As such the system should have sufficient strength to avoid such occurrences under full ultimate wind pressures or ultimate limit state earthquake applied either as a face load or as in-plane interstorey drift. Because of the uncertainties involved, the glazing system should be capable of sustaining 1.33 times the calculated inelastic interstorey drift.

D7 Aim: To prevent crack development within concrete or clay masonry walls when they are subjected to in-plane racking distortion. The design action that induced such distortions may be either serviceability intensity wind or earthquake forces. Cracks as a result of such actions are likely to manifest themselves as either a single horizontal crack which develops along a single mortar course, or as intermittent tension cracks within walls of a high aspect ratio. Since masonry panels are generally strong (but often brittle) within their own plane, such cracking is not usual. Indeed the unforeseen presence, rigidity and strength of infill masonry panels has often been attributed as being the cause of short column shear failures under moderate earthquake attack, where the masonry infill effectively and dramatically reduces the column curvature, resulting in very high shear stresses within the column. Separation of the infill panels from the structural frame is recommended as an effective control measure both to avoid premature cracking and short column effects.

Moisture and temperature related shrinkage of masonry panels are perhaps the most common causes of cracking. Such strain-related cracking is to be expected adjacent to openings where there is a large change of section. The provision of purpose-made shrinkage control joints in such locations is usually sufficient to prevent uncontrolled cracking and the resulting serviceability failures. Cracks commonly develop along a single mortar joint as in-plane shear is applied. Although such panels are usually strong and rigid, they are largely intolerant to in-plane racking and brittle shear sliding along such a crack is commonly the failure mechanism.

Differential settlement within the foundation or support beams beneath masonry walls will also induce strain into masonry wall panels. The inherent in-plane strength and rigidity of the wall itself, whether reinforced or unreinforced, usually enables the wall to be self-supporting between distant points of support. In such cases horizontal cracking and separation is likely to develop within the lower mortar joints, commonly at the damp proof course where present. Water ingress can result, but will only be of concern where such water can migrate further into the structure (refer also D15 below). Where the masonry panel cracks from other causes or where a crack control joint has been built into the panel, then the wall segments on either side of the crack will act as independent rigid bodies. In such cases the support mechanism beneath the masonry will become loaded, but the geometric interaction of the rotating panel segments will continue to influence the overall system deformation.

D8 Aim: To prevent crack development within masonry walls when subjected to actions normal to their plane. Face load pressures can be applied either by wind or earthquake actions. The first crack within the masonry will usually form near the base of the panel. This occurs at a very low strain (<0.001) when the tensile capacity of the mortar is exceeded. Re-entrant walls usually provide support to each other around corners and seldom experience cracking of this nature. Displacement compatibility usually results in inclined diagonal cracks extending from the lower level base crack upwards towards the supported corners. Whilst the occurrence of the base crack is seldom a serviceability failure (i.e. will not generally require repair nor cause secondary damage), the development of the diagonal cracking is unsightly and should be avoided. It is upon this limit that the acceptable response limit has been based.

Within reinforced structural masonry the reinforcing steel will thereafter provide the tensile component to the ongoing overturning moment. Likewise, cracks that develop during the application of the design action are of little importance. Rather it is the residual cracking which remains following the removal of that action. For reinforced masonry, residual crack widths less than 0.5 mm, although visible on smooth, light coloured surfaces, seldom cause problems. Cracks of this size can be expected when the steel strain reaches 0.004 when the design action is applied (i.e. some inelastic steel strain has occurred).

For unreinforced masonry veneer, the initial base crack generally closes upon removal of the applied design action under the self-weight of the panel. Such panels are now free to rotate at their base. When the supporting framing is 'flexible' compared to the panel itself, then the panel will largely be self-supporting, rocking along its base and supported by veneer ties at or near the top of the support framing where the ceiling or floor diaphragm provides support. Only following the onset of a second crack within the body of the veneer panel is a mechanism formed. At this stage the load pattern changes dramatically. The substrate now fulfils its role and supports the veneer through the mid-height ties. The characteristics of earthquake induced ground motion also changes with the development of the initial base crack. Sliding along the crack face (both in-plane and out-of-plane) limits direct transmission. The veneer solely contributes to the seismic mass, experiencing excitation indirectly through the response of the structure itself.

D9 <u>Aim</u>: To prevent <u>crack development within plaster or gypsum based wall boards</u> when subjected to in-plane racking. The acceptable response limit has been established during wall panel tests on bracing panels. Cracks in the wall linings usually develop at or near door or window lintels where there is a gross change in the section of the lining. Within unpenetrated lined panels, in-plane shear distortions of up to 12 mm (per 2.4 m height) can be sustained without significant cracking.

D10 <u>Aim:</u> To prevent <u>damage or excessive deformation to plaster or gypsum based sheet lining boards</u> when subjected to actions normal to their plane. The acceptable response limit roughly coincided with the loss of adhesion (due to paper delamination) in the vicinity of glue-fixing daubs. When sheets are all mechanically fixed (eg. screws or nails) the acceptable response limit can be relaxed to span/100, which is roughly coincident with the initial onset of mid-height cracking typically at the plaster joints.

D11 Aim: To prevent damage or excessive deformation of plaster or gypsum lining boards (when affixed to framed walls) when they are subjected to normal occupancy soft-body impact. The design action imposed has been simplified to a horizontally applied concentrated point load (which may be considered as being applied over a 300 mm diameter area). Such an action has been assessed as being equivalent to the deformation caused by people falling against the wall lining. The stipulated acceptable response is stated as the deflection to height ratio beyond which the rigidity of the glue is known to induce delamination of the facing paper. The onset of damage can be difficult to quantify, particularly as this may develop within the core of the sheet, and will only become noticeable when the face is redecorated (i.e. the surface is moistened). Punching shear failures consequential to hard-body impact or from other causes are also considered to be serviceability failures. Where the normal occupancy use is reasonably likely to experience such impacts, the lining material should be selected so as to have sufficient impact resistance that the probability of damage is minimised. This is difficult to predict by design, and verification of adequate punching shear resistance using an experimental method is recommended.

D12 Aim: To prevent damage or excessive deformation of panelised internal partition systems when subjected to normal occupancy soft-body impact. This control provides a guide of acceptable impact resistance of inter-office partitions. Panelised systems such as these are typically mechanically clipped together. The resulting surface is usually interrupted by the link clips with complete sheets being individually framed. Furthermore occupants are generally more tolerant to deformation within an office environment when compared with a domestic environment. The limit of acceptable response is thus more liberal for such panels when considering the onset of non-structural damage.

D13 Aim: To prevent damage to walls with other (non-specific) linings when subjected to actions normal to their plane (i.e. flexural deformation). This criterion is very general, in that some systems within each group of ‘other systems’ will be more tolerant to movement than others. Whilst the criterion should be adequate, where ever possible, the deformation at which there is an onset of damage should be ascertained for each system.

D14 Aim: To prevent the onset of damage to wall and roof sheathing materials which are supported by portal frames. The criterion was developed to avoid excessive tearing of either the wall or roof sheathing. The specific instance envisaged relates to the roof membrane between portals which have markedly different lateral stiffness (e.g. over the end bay of a multibay portal system where the external sheathing or end wall bracing stiffens the exterior bay while the interior portals, which rely on moments actions remain relatively ‘flexible’). The resulting shear distortion imposed on the roof membrane will tear the cladding around its fixings. Likewise, the wall sheathing covering the end wall must either possess sufficient strength to accept the lateral shear imposed upon it, or be able to accept distortion compatible with the end-bay portal without excessive tearing at the fasteners.

Weathertightness problems will result where such tearing extends beyond the cover of the fixing washer. Where upper floors are supported from the portal frame, side sway control (refer S15 & S16 above) and/or internal partition deflection controls will also need to be considered.

D15 Aim: To ensure the foundation and support beams are sufficiently rigid to avoid cracking within supported masonry wall systems. The onset of cracking within veneer or structural masonry walls is discussed under D8 above. The design action imposed for this criterion is the dead and short-term live load active on the floor slab supported by the beam under consideration. Provided adequate shrinkage controls are in place within the masonry panels, the wall mass supported by the beam may be considered as that mass beneath a wall segment contained within a triangular segment of wall where the inclination of the triangle is 30° from each support point.

D16 Aim: To prevent plasterboard lined walls against in-plane vertical deformation resulting from the relaxation of the support beams. Deep beam actions are commonly very effective in such walls, provided the wall panels are not penetrated by door openings. The control applies only to load-bearing walls where there is sufficient vertical load being imposed so that the panel is unable to sustain the load itself and therefore remains in contact with the supporting floor.

4. THE STRUCTURE & LAYOUT OF THE SERVICEABILITY TABLES

This section explains the rationale for the presentation style used in Table 1. Designers are expected to consider the actions imposed on the elements under consideration and the reason for limiting deformations and to nominate the appropriate limit for their specific circumstance. Seldom will all limits be applicable. Only those which apply in each specific instance need to be satisfied.

The verification process involves determining, either by calculation or by test, the response of the system when subjected to the Imposed Design Action stipulated (column 5), and verifying that this response is less than the Limit of Acceptable Response specified (column 6).

4.1 Control Phenomenon (Column 1)

4.1.1 Sensory: seen or felt

The primary entry point for Table 1 is the serviceability phenomenon for which the limit is being considered. This first part of the table addresses response phenomenon which are sensed (i.e. either seen or felt). Such observations are generally highly subjective and the boundaries between acceptable and unacceptable response are quite indistinct. People's senses are usually triggered by some secondary cue (e.g. rattling of crockery) or distorted reflection (gloss paint finishes or mirror effects from windows – particularly at night). Removal of the cue changes the acceptable deformation limit. The presence of such cues is difficult (if not impossible) to ascertain during design, and often a conservative assessment is required. The nature of the activity being undertaken within the space will also influence the acceptance limit. 'Busy' activity areas are much more accommodating than those where 'tranquil' events occur.

Observed effects usually involve the application of long-term loads. Cambers can be used to reduce sag with the camber generally being equated to the deflection resulting from long-term load (self-weight plus long-term live load) together with any creep effects if present. When cambers are present, only the additional deflection imposed by the incremental live load should be considered for compliance with the acceptable limits stated.

Felt effects are usually short term and often dynamic. People are much more susceptible to dynamic or transient movement, particularly if they are unexpected or if they continue over a significant period. The threshold of acceptability varies with the sensitivity of the observer, the response frequency, the particular activity within the space, the time of day and the duration of vibration.

4.1.2 Functionality

Excessive deflections may be unacceptable when they interfere with the use of the building. This may show up as annoyance to the users, or the inability of the structure to perform as intended.

Problems with stormwater run-off from very flat, exposed surfaces (roofs or decks) can be exacerbated by sag induced by water ponding on flat roof surfaces. This, combined with flawed flashing details can lead to rainwater ingress into interior spaces and consequential accelerated damage and deterioration of fittings and furniture. Water ponding on flat external surfaces can render them unusable as outside spaces for long periods of time because of the formation of puddles after a rainfall. The ability to operate large doors and windows, or to refit demountable partitions in alternate locations, may be limited by excessive deflections.

Some floors, such as bowling alleys, gymnasiums or those supporting particularly sensitive equipment, can be rendered inoperative unless more stringent serviceability controls are applied. Such floors are much more intolerant to undulations and can be rendered inoperable when normal tolerances are applied.

4.1.3 Prevention of damage to secondary elements

Non-structural or secondary elements are required to be sufficiently robust to ensure they are undamaged when subjected to normal, frequently occurring actions. Such actions may be applied directly to these elements (roofing systems under maintenance loads) or may be as a consequence of insufficient clearances between structural and non-structural elements. The criteria contained within this section are intended to prevent the premature onset of damage to these elements thereby ensuring they may continue to function without repair.

Walls, partitions, windows, ceilings, floor and roof coverings, facades, service pipework, lifts and stairs can all experience damage in the form of cracking, buckling, tearing, folding or wrinkling as they distort under unexpected loads. Such actions may render the building unserviceable by damaging functional aspects of the structure (i.e. weathertightness, thermal and sound insulation), or by damaging it aesthetically. Specific measures can be taken to avoid loading non-structural components, either by separating them from the structure or by other means. The separation must be sufficient for the actual movement and the movement details must be effective thereby truly separating the non-structural elements. In such cases, the serviceability drift limits need only be applied to those elements not adequately separated or to conditions when deformations exceed the separation provided.

Non-structural elements are normally installed after initial self-weight induced deformations have occurred. Long-term creep and short-term live loads are usually the imposed design actions. Generally secondary elements are considerably more tolerant to movement than has previously been thought. Although hard data is again sparse, work undertaken by Thurston (1993) and others indicates that even brittle elements such as windows or gypsum stopped plasterboards are able to withstand moderate movement without damage. Masonry elements, whilst experiencing crack development with very little movement, continue to perform as effective claddings until major cracking is experienced. This does not usually occur until significant movement is present. In-plane and out-of-plane behaviour of such systems differ, this being reflected by different criteria.

4.2 Ref Id (Column 2)

This column comprises a letter and a sequential number. The letter relates to the control phenomenon which forms the basis for the consideration. The combination of letter and number provides the link between Table 1 and Table 1A.

4.3 Element Being Assessed (Column 3)

Most elements will be repeated within each control phenomenon category. They are listed in each category from the roof downwards. The designer is expected to consider whether the serviceability aspect under consideration (column 4) is applicable to the element being considered within the specific structure being designed. There will be many design cases where control of a specific control is unnecessary for a given element, in which case the criterion may be ignored.

4.4 Serviceability Aspect under Consideration (Column 4)

The particular aspect of the element which has been considered when prescribing the criterion is listed within column 4 (e.g. sag, ripple, sway, ponding, etc). In some instances this specific attribute may either not apply or be unimportant. In such cases the stipulated criterion should be ignored.

4.5 Imposed Design Actions (Column 5)

The actions, both with regard to the loading intensity and the combination of loads applied, are described in Section 5 below. The duration of the applied action must be realistic so that creep or other time dependent characteristics can be assessed. Load combinations, complete in both location and direction, should be applied so as to ascertain the least advantageous action on the element.

Where long-term effects are considered, the applied live loads should reflect the appropriate duration of application for the phenomenon being controlled. Sometimes the response of an element may be less severe because of the presence of greater mass (such as floor vibration effects). In such cases it is appropriate to reduce the live load component to half of the long-term live load (i.e. $| * \Psi_L * Q$). For pseudo-impact assessments (e.g. items S14 and S15), the full intensity nominated concentrated point action, Q, is to be applied.

4.6 Limit of Acceptable Response (Column 6)

Acceptable limits for each element within each category are listed as a span or height ratio (i.e. span/360). For horizontal elements (e.g. floors, lintels, beams, etc) this relates to the member ‘span’ and for vertical members (e.g. columns, walls, windows, etc.) to the element ‘height’.

5. DETERMINING APPLIED ACTIONS

5.1 General

The intensities of the loads and forces and the factored combinations thereof which are to be used as the basis for design of New Zealand buildings are prescribed in the NZ Loadings Standard, NZS 4203 [SNZ, 1992]. The loading symbols, intensities and combinations used in this report are consistent with those specified in that standard.

6. ASCERTAINING ELEMENTAL DEFLECTION

6.1 General

Simple deflection formulae are presented in many engineering handbooks and manuals, with perhaps the most popular being the "Steel Designers Manual" [Constrado 1983] and "Formulae for Stresses and Strains" [Roake 1975]. Whilst such formula provide analytical tools to enable designers to assess deflections, the transition from idealised to real elements, loadings, dimensions and compound deflections must be considered. Designers should be particularly alert to compound deformations where the displacement of primary elements results in support deformation of secondary elements. The secondary element thus experiences both linear and possibly magnified rotational deformation in addition to its own static (or dynamic) deformation.

When determining the deflection of structures or building components there are many factors which should be assessed:

a) Sensitivity of realistic (actual) modulus of elasticity (refer Section 6.2)

b) Section modulus (refer Section 6.3)

c) Changes in section (notches, holes, cut-outs, composite sections, etc.)

d) Component end restraint and rotation effects (refer Section 6.4)

e) Flexibility of supports (combined deformation, rotation effects) (refer Section 6.5)

f) Environmental effects (thermal expansion/contraction, moisture movements & shrinkage) (refer Section 6.6)

g) Duration of load (creep effect) (refer Section 6.7).

Table 2 has been prepared after Rainger [1984]. It details the range of material properties which will commonly influence deformation-related serviceability limit states compliance for many building materials.

Table 2: Typical building material deformation properties (from Rainger [1984] & BRANZ [1991])

(Notes: 1 based on extreme (but not saturated) wet to extreme dry; 2 based on change from green to 12% mc)

Material type	Modulus of Elasticity, E x 10^3 MPa	Linear Thermal Expansion coefficient α_t (mm / m / °C)	Irreversible Moisture Expansion coefficient α_m (% change per % change in moisture)
Cement -Based Composites			
Cement mortar	20 - 35	0.0010-0.013	0.02 - 0.06
Normal concrete	$3.32\sqrt{f_c} + 6.9$	0.010-0.014	0.02 - 0.06
Concrete blocks	15	0.006-0.012	0.02 - 0.04
Cellulose cement sheets	14 - 26	0.008-0.012	0.15 - 0.25
GRC	20 - 34	0.007-0.012	0.15 - 0.30
Clay			
Tiles	20 - 34	0.005-0.008	
Bricks	20 - 34	0.005-0.008	0.01 - 0.05
Metals			
Aluminium alloys	73	0.024	No effect
Brass	100	0.018-0.021	"
Cast iron	80 - 120	0.01	"
Mild steel	210	0.012	"
Copper	95 - 130	0.017	"
Lead	14	0.030	"
Stainless steel	200	0.018	"
Zinc with rolling	140 - 200	0.03	"
without rolling		0.07	"
Natural Stone			
Granite	20 - 60	0.008 - 0.010	0.01 - 0.015
Limestone	10 - 80	0.003 - 0.004	0.01
Marble	35	0.004 - 0.006	No data
Slate	10 - 35	0.009 - 0.011	No data
Rubbers & Plastics			
Asphalt		0.03-0.08	No effect
Acrylic	2.5 - 3.3	0.03 - 0.09	"
Epoxy		0.05-0.09	"
Glass reinforced plastic	6 - 12	0.02-0.035	"
Neoprene		0.19-0.21	"
Polycarbonate	2.2 - 2.5	0.06-0.07	"
Ploypropylene		0.08 - 0.11	"
Polyethylene		0.11-0.2	"
Polystyrene		0.06 - 0.08	"
PTFE		0.05 - 0.1	"
PVC	2.1 - 3.5	0.04 - 0.07	"
Glass			
Plain untinted	70	0.009-0.011	No effect
Wood & wood-based products			
Hardwoods	12-21 Dry - 9-16 wet	0.004 - 0.006 with grain 0.03 - 0.07 cross grain	0.8 (1) ~ 9.5 (2) Tangential 0.5 (1) ~ 4.0 (2) Radial
Radiata pine	9.0 Dry - 5.8 green	0.004 - 0.006 with grain 0.03 - 0.07 cross grain	3.9 Tangential 2.1 Radial
Douglas Fir		0.004 - 0.006 with grain 0.03 - 0.07 cross grain	4.9 Tangential 2.8 Radial
Particlebd width & length thickness	6 - 10	0.03 - 0.04	0.3 - 0.55 (65% - 95% rh) 8.0 - 15.0.
Hardboard width & length thickness	4 - 6	0.010	0.1 - 0.2 7.0 - 9.0
MDF width & length thickness		No data available	0.25 - 0.35 11.0 - 13.0
Plywood with grain cross grain	6 - 12	0.02	0.15 - 0.2 0.2 - 0.3
Softboard . width & length		No data available	0.1 - 0.2 (50% - 90 % rh)

thickness	8.0 - 10.0

6.2 Modulus of Elasticity (E)

The modulus of elasticity, E, is the ratio of the stress to strain of a material while it deforms elastically. Whereas some materials have a reasonably consistent modulus of elasticity (e.g. steel), other materials (e.g. cement products or plastics) will have a modulus which is influenced by the composition of the element, the level of stress present, and the temperature and moisture content fluctuations it experiences in service. For reinforced concrete systems, E is proportional to $\sqrt{f'_c}$. NZS 3101 [SNZ 1995] prescribes the value of f'_c used in design and is defined as the 28 day compressive strength. It is verified by tests, with the material specification usually demanding that 95% of the samples tested are to be in excess of the nominated value. Thus the actual compressive strength and hence the modulus of elasticity will usually be between 30 and 40% greater than that indicated in Table 2 for concrete products. Long-term static load deflection calculations sometimes accommodate the effect of creep by modifying the value of E. (Refer Section 6.7).

6.3 Effective Moments of Inertia (Second Moment of Area) (I)

The second moment of area, I, is a geometric property of the section. Flexural deflections are generally inversely proportional to I. Sections of materials which are homogeneous (e.g. steel or timber) have a second moment of area which is the same whether the curvature of the element is positive or negative. Some structural materials, such as reinforced concrete or glass-reinforced cement products, are engineered to utilise specific favourable attributes of each material within a composite to increase overall section efficiency. An equivalent transformed section approach is usually necessary when assessing the second moment of area of such composite systems. This involves developing an effective sectional area based on the EI value of each component. When cracking is present the effective section may change markedly. Variation of moment along an element and the presence of high axial compression both influence the depth of cracking present and these effects may be considered in highly refined design procedures. For reinforced concrete sections, Table C3.1 in the commentary to the concrete design standard, NZS 3101 [SNZ 1995], recommends the extent of cracking which should be considered and is reproduced here as Table 3.

6.4 Effective End Restraint

The degree of restraint necessary to achieve a fully restrained element is usually difficult to attain, particularly within a framed system. The designer is best to anticipate partial end fixity only and assess beam deformation accordingly. For squat beams of short span, the shear deformation is likely to contribute a similar deformation to flexural. Although such beams are usually stiff and serviceability is seldom an issue, deflection calculations for such systems should include the contributions from both shear and flexure.

When considering the dynamic response of beams or floor systems which are continuous over several near-equal spans, the inertial mass within adjacent spans may actually contribute to the dynamic response of continuous beams and the inertial mass should be considered as moving within the direction of motion rather than providing end restraint, as is commonly assumed for static stiffness considerations.

As most simple bending deflections are generally proportional to the third or fourth power of the member span, it is important that the member span be realistic. It is often appropriate to make some allowance for end-zone rotation by increasing the effective beam length some distance (say 20% of the column width) into the face of supporting columns (refer NZS 3101 [SNZ 1995]).

Table 3: Suggested effective concrete section properties (after NZS 3101:1995)

Type of Member	(1) Ultimate Limit State	Serviceability Limit State (2) $\mu = 1.25$	(3) $\mu = 3$	(4) $\mu = 6$
1. **Beams**				
Rectangular	$0.4\ I_g$	I_g	$0.7\ I_g$	$0.40\ I_g$
T or L beams	$0.35\ I_g$	I_g	$0.6\ I_g$	$0.35\ I_g$
2. **Columns**				
$N^*/f'_c A_g > 0.5$	$0.8\ I_g$	I_g	$0.9\ I_g$	$0.80\ I_g$
$N^*/f'_c A_g = 0.2$	$0.6\ I_g$	I_g	$0.8\ I_g$	$0.60\ I_g$
$N^*/f'_c A_g = -0.5$	$0.4\ I_g$	I_g	$0.7\ I_g$	$0.40\ I_g$
3. **Walls**				
$N^*/f'_c A_g = 0.2$	$0.45\ I_g$, $0.8\ A_g$	I_g, A_g	$0.7\ I_g$, $0.9\ A_g$	$0.45\ I_g$, $0.8\ A_g$
$N^*/f'_c A_g = 0$	$0.25\ I_g$, $0.5\ A_g$	I_g, A_g	$0.5\ I_g$, $0.75\ A_g$	$0.25\ I_g$, $0.5\ A_g$
$N^*/f'_c A_g = -0.1$	$0.15\ I_g$, $0.3\ A_g$	I_g, A_g	$0.4\ I_g$, $0.65\ A_g$	$0.15\ I_g$, $0.3\ A_g$
4. **Coupling beams**				
Diagonally reinforced	$\dfrac{0.4I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$	$\dfrac{I_g}{1.7+1.3\left(\frac{h}{L}\right)^2}$	$\dfrac{0.7I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$	$\dfrac{0.4I_g}{1.7+2.7\left(\frac{h}{L}\right)^2}$
Conventionally reinforced	$\dfrac{0.4I_g}{1+8\left(\frac{h}{L}\right)^2}$	$\dfrac{I_g}{1+5\left(\frac{h}{L}\right)^2}$	$\dfrac{0.7I_g}{1+8\left(\frac{h}{L}\right)^2}$	$\dfrac{0.4I_g}{1+8\left(\frac{h}{L}\right)^2}$

Where I_g = uncracked section second moment of area (mm^4)
A_g = uncracked section area (mm^2)
h = height of coupling beam
L = length of coupling beam (mm)

μ = structural ductility factor
N* = axial load
f'_c = 28 day concrete compression strength

6.5 Flexibility of Supports

Simple deflection formulae generally do not include any allowance for settlement or movement of supports, yet this is the most common cause of deflection problems. Overhanging cantilever beams which are supported on flexible supports are particularly prone to problems which relate to the compound free-end deflection resulting from deformation of the support and flexural behaviour of the overhanging portion of the beam. Often the free end of the cantilever supports a parapet or hand-rail allowing the end rotation of the beam to be translated into misalignment of the hand-rail (both vertically and horizontally). In such cases three-dimensional modelling techniques are recommended, with tight tolerance on the movement acceptable by the support element.

6.6 Environmental Influences on Deflection

All materials experience dimensional changes with variations in temperature. Many also experience dimensional changes with variations in moisture content. Many also experience strength loss and stiffness degradation because of aging and weathering. The magnitude of each of these effects varies widely between materials. Several such effects are recoverable with the property reverting to, or near to, its original state on the reversal of the temperature or moisture content change. Aging effects are obviously unidirectional with the 'recovery' phase absent. For any given overall dimensional change, a proportion will rebound when conditions revert back to their original state. Most materials also exhibit a proportion of irrecoverable change which may be incremental change if cyclic variations apply.

As the thermal and moisture movement effects are usually in opposition to each other (i.e. a temperature rise expands the component by thermal expansion, but over time removes moisture resulting in shrinkage of the component), it is often practical to design with the lesser parameter held constant while the effects of the second are considered. The time related aspect of the evaluation can be significant in that thermal effects tend to be immediate while changes in section moisture content are usually delayed.

Movement within the building fabric may be accommodated in several ways as follows:

i) Control joints permit the relief of internal strains within elements. Provided they are installed at sufficiently close centres, the strain (and hence related internal stresses) within the elements will remain small and cracking will be avoided. When selecting the location of control joints the designer is encouraged to include other changes of section (openings or penetrations) and also changes in the alignment of the element and to locate the control joints so they work with rather than against the natural movement of the element itself.

ii) Where practical, it is often better to allow movement to occur rather than to constrain elements. Elements which are rigidly constrained build up internal stresses which they either impart to their support constraints or seek relief through the generation of cracks.

iii) In elements where movement is particularly critical the designer may choose to use alternative materials with lower thermal or moisture expansion characteristics, or may minimise the exposure of the element to the agent through the use of protective coatings, thereby minimising the extent of moisture take-up or the in-service temperature range.

Further discussion on these approaches and some recommendation as to appropriate expansion or contraction details are presented by Rainger (1984).

6.6.1 Thermal movements

Materials expand when heated and contract when cooled when exposed to New Zealand conditions. The movement is proportional to the temperature variation of the material. Any temperature change due to solar effects will depend on the colour, and the inclination and exposure of the surface to the sun.

Temperature related data presented in Table 4, Table 5, and Table 6 has been taken from BRANZ Bulletin 285 "Designing for Thermal & Moisture Movement" [BRANZ 1991]. The measurements were derived for New Zealand conditions and further background information is presented within that bulletin.

Table 4 shows how different colours affect the maximum temperature of galvanised steel roofs. An alternative approach, as shown in Table 5, indicates how materials build up and lose heat compared with the ambient air temperature through solar gain or night loss. The effect of inclination and exposure of the components is not incorporated. When used in conjunction with Meteorological Service temperature records, these tables provide a method of assessing temperature extremes. The coefficients of linear thermal expansion for many materials are listed in Table 2. For convenient design application, the movements (mm) per metre length are listed for these materials when subjected to a 60°C temperature variation. The 60°C temperature range should be considered as a guide only. For very dark, well-insulated panels, the in-service range of temperature may be significantly more than 60°C, while light-coloured, well-ventilated panels will vary experience much lower variation as indicated in Table 4.

Table 4: Guide for surface temperature variation for uninsulated roofs (from BRANZ 1991)

<table>
<tr><td>Surface Colour</td><td colspan="2">Roof Surface Temperature °C</td></tr>
<tr><td></td><td>day
for a 20 °C air temp.</td><td>night
for a 0 °C air temp.</td></tr>
<tr><td>Galv. steel black</td><td>70</td><td>-10</td></tr>
<tr><td>Galv. steel red</td><td>63</td><td rowspan="5">between
-5 and
-10</td></tr>
<tr><td>Galv. steel unpainted (zinc)</td><td>60</td></tr>
<tr><td>Galv. steel alum. (paint)</td><td>50</td></tr>
<tr><td>Galv. steel cream</td><td>48</td></tr>
<tr><td>Galv steel white</td><td>44</td></tr>
<tr><td>concrete</td><td>30-35</td><td>-5</td></tr>
</table>

Table 5: Estimated extreme temperatures on buildings

Type of Surface	Surface Temperature Variation °C from Ambient	
	Above	Below
Dark roofing	20-40	10
Steel and other metals	15-25	5-10
Concrete and masonry	10-15	5

(Note: the temperature of dark roofs may be between 20 and 40°C above ambient air temperature during the day, and up to 10° C below ambient temperature at night.)

Table 6: Temperature ranges for some building forms in New Zealand (from BRANZ 1991)

Building Element	Extreme Temperatures °C	
	Maximum	Minimum
Precast concrete or exposed concrete eaves or edges of a floor slab or light coloured masonry wall	50	-10
As above but dark coloured	65	-10
Dark coloured glass or ceramic tiles or metal with insulation behind	80	-15
As above but light coloured	60	-15
Black metal panel exposed behind clear glass with insulation behind panel	130	-5
Clear glass in front of dark insulated background such as the panel described above	80	-15
Aluminium curtain wall mullion (white or natural colour)	50	-5

The designer should be aware that the temperature of various materials may also influence other properties such as strength and ductility.

Thermal movement can be calculated from equation (1) below.

$$\Delta l_t = \alpha_t \, . \, . \, \Delta_t \, . \, L \qquad (1)$$

where Δl_t = change in length due to temperature variation (m)
α_t = coefficient of linear thermal expansion (mm/m/°C)
L = original dimension of component (m)
Δ_t = change in temperature (°C)

(Refer to column 3 of Table 2 for values of α_t)

Thermal movement example:

Q. For a cream coloured steel long-run roofing sheathing – calculate the movement allowance required to prevent the onset of buckling and/or possible tearing at the fixing points.

A.

Nominal length of roofing	11 m	given
Daily range of surface temperature	+48 - (-10) = 58 °C	(from Table 4)
Thermal movement	0.012 mm / m / °C	(from Table 2)

Then calculating the maximum movement
= length (m) x temperature range x thermal movement coefficient
= 11 x 58 x 0.012
= **7.6 mm** variation in length should be allowed for such a roofing system over its operating life.

6.6.2 Moisture movements

Moisture movements result from porous materials absorbing and releasing moisture. Moisture related movement can be either irreversible or reversible.

Irreversible shrinkage usually occurs shortly after manufacture and is most commonly related to drying. Examples of this action are most readily seen in concrete or cement-based products. About 70% of the total drying shrinkage of such products occurs within three months of manufacture.

Irreversible expansion of clay bricks and tiles is not uncommon in the UK and Australia. Because of differences in the chemical composition of the clay used for most New Zealand brick, it has not generally been an issue in the same items manufactured in New Zealand. With the increased use of imported clay products over recent years however, it is becoming increasingly important to include expansion joints within New Zealand veneers to allow for such movement. Guidance is given for concrete masonry veneers within NZS 4223 “Standard for Masonry Design & Construction” [SNZ 1990] where it is generally prescribed that expansion joints should be provided in walls at not more that 6 m centres, and at either one (for opening <2.4 m) or both veneer parapets. These measures would also be appropriate for clay masonry units.

Reversible moisture movements occur in some materials which are hygroscopic (i.e. absorb and release moisture). Variations in the relative humidity of the host environment are, commonly, sufficient to initiate such effects. Timber is the most common material to exhibit this behaviour. Within the cellular structure of timber, moisture is able to be stored both within the cell and within the fibres forming the cell walls. The timber reaches its fibre saturation point (FSP) when all free water within the cell cavity is removed while the walls hold the maximum amount of bound water. This is commonly at a moisture content of about 30% for radiata pine, eucalyptus, tawa, silver beech and kahikatea, and 25% for macrocarpa, matai, red beech Douglas fir, redwood kauri and rimu [BRANZ 1991]. Above FSP, moisture content has no effect on the external dimensions of the timber. Below FSP, shrinkage will occur as the structure of the fibre walls change with moisture content. Such movement is reversible in that the addition of moisture results in expansion to similar physical dimensions. The extent of dimension change varies with orientation to the grain. For radiata pine the tangential and radial moisture content related shrinkage is 39 and 21 mm/m/% change in mc respectively. When assessing dimensional changes, the variation in moisture content below FSP is required to be considered. The in-service moisture content of the timber is dependent on the environmental conditions, specifically the relative humidity to which it is subjected. Moisture related problems manifest themselves when the steady state relative humidity reaches or exceeds 80 to 85%. Under these conditions radiata pine often reaches a moisture content of about 25% and is likely to experience mould growth, fastener corrosion and other deterioration characteristics. Typical timber moisture contents more commonly encountered in service are:

i) for a well maintained mechanically heated internal swimming pool (20%)
ii) for normal residential framing (10-12%)
iii) for an air conditioned office (5-7%).

Wherever practical, timber should be installed at or near the in-service equilibrium moisture content for the particular end use, and measures should be taken to ensure this is maintained both during construction and during service. This will minimise movement within the framing and minimise serviceability problems such as cracking and peaking or popping of lining systems or their fixings. In addition, wall-framing elements should remain straight and true prior to lining, thereby minimising problems which have traditionally accompanied twisting and warping of framing timbers, particularly when smaller timber sections are used.

Moisture movement can be calculated using equation (2):

$$\Delta l_m = \alpha_m \, . \, \Delta_m \, . \, L \qquad (2)$$

where Δl_m = change in length due to moisture movement (m)

α_m = moisture movement coefficient (% elongation per % change in m.c.)

L = original dimension of component (m)

Δ_m = change in moisture (%)

(Refer to Table 2 column 5 for values of α_m)

6.7 Long Term Deflection: Creep

6.7.1 Creep in timber

The creep mechanism in timber is related to the distortion of the cellular structure of the timber which accompanies the application of load. Moisture movement within the wood cells accentuates such distortion. The ‘long-term’ distortion is related to the application of ‘long-term’ loading. Thus the dead load and the sustained portion of the live load need to be considered as contributing to creep effects.

The effect of creep is incorporated into the design process by modifying the modulus of elasticity (E), and hence the elastic deflection, by the K_2 factor of Clause 2.6.2.1 of NZS 3603 Structural Timber Design (SNZ 1993), which is applicable to the duration of the load being considered. As the amount of creep is related to the moisture content, so changes of moisture content (i.e. the movement of water in and out of timber) will accentuate the distortion. Where the moisture content varies cyclically by more than a few per cent, as would be the case when the timber is in an exposed environment, the modification factor (K_2) may need to be increased. This increase is likely to be between 30% and 50% greater than those values nominated.

6.7.2 Creep in reinforced concrete

The mechanism of creep in concrete is related to changes in the structure of the aggregate and cement matrix. Because the process is dependent on several variables (e.g. age at which the concrete is loaded, duration of loading, relative humidity, mix proportions and aggregate, section geometry) NZS 3101 “Structural Concrete Design” (SNZ 1995) suggests that long-term deflections (for creep and shrinkage to be accounted) be assessed by magnifying the immediate deflections caused by the dead load and sustained live load (i.e. $G + \Psi_L Q$) by a factor not less than K_{cp}=(2-1.2(A's/As))> 0.6 where A's and As are the compression and tension steel areas respectively. The standard recognised that studies showing creep may vary by up to

400% depending on the aggregate used. The suggested values of K_{cp} have been plotted in Figure 1 (SNZ 1995).

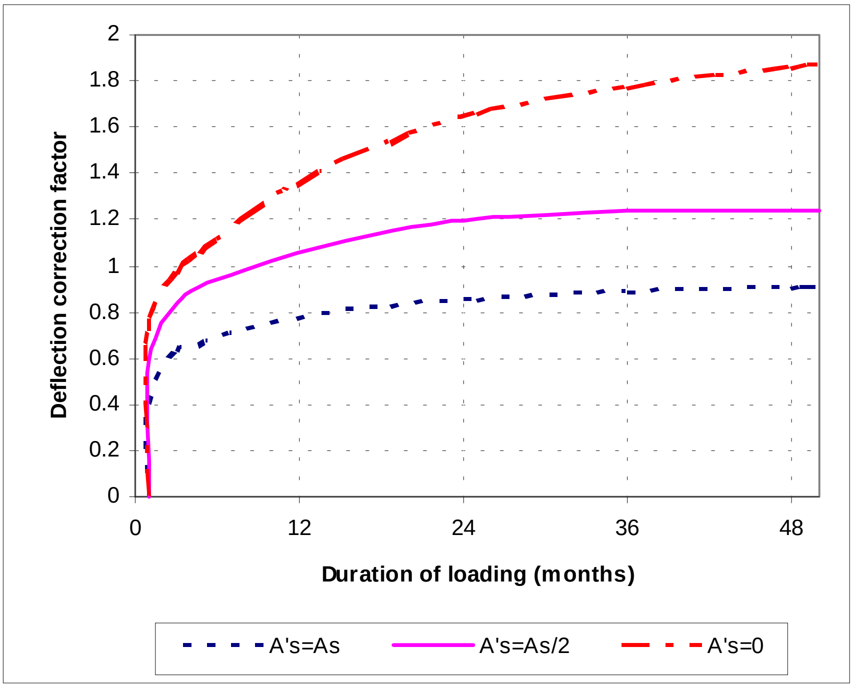

Figure 1: Multiplier for long term concrete deflection, Kcp

7. DYNAMIC EFFECTS

7.1 Problems with Vibration

Building vibration effects that result in serviceability problems are often classified by the duration over which they occur. Long-term vibrations are classified as ‘continuous’, while short-term effects are known as ‘transient’. The cause, the acceptance criteria and the treatment for each are different.

Continuous vibrations arise from periodic forces which continue for a significant period of time (e.g. machinery, oscillating equipment, or certain rhythmic human activities such as dancing). Where the frequency of the periodic forces is synchronised with one of the element’s natural frequencies of vibration, resonance may develop (see Section 7.4). Resonant response can greatly magnify vibration effects, to the extent that structural damage, overloading, and, in the extreme, structural collapse can result. Likewise, fatigue prone materials will experience the onset of this undesirable phenomenon earlier as the internal strains associated with resonant response will be equivalent to a higher effective stress (consequential to resonance being present). Resonant behaviour is not affected by the level of structural damping present. Mitigation measures commonly involve separating the excitation source from the response element, usually by incorporating some form of isolation technique. While many continuous vibration problems (e.g. those generated by oscillating plant or machinery) can be treated in this manner, there are others where isolation is not practical. Excitation generated by rhythmic crowd response is one circumstance where

the excitation source (i.e. the crowd) cannot easily be separated from the response element (ie the floor or building).
In these cases the frequency of the forcing function (or harmonics thereof) should be widely disparate from the natural frequency of both the building and the supporting components to ensure resonant effects are avoided. A design procedure for this type of action is outlined in Section 7.4.

Transient vibrations can be caused by intermittent excitation sources which often include a spatial variance component. People walking or running over a floor are one common example of a transient excitation source. These impulses generate a combination of responses within the floor system which include quasi-resonant effects and direct impulse response. Problems encountered from transient vibrations are usually annoying rather than structurally damaging (i.e. rattling of china, felt motion, or rocking of tall furniture). Some amplification is common when the supporting element responds in harmony with the excitation source. However, with a mobile source, and where spatial variations are present within the response mechanisms, true resonant effects are avoided. In contrast with a truly resonant effect, transient vibration problems are greatly influenced by the degree of damping present within the responding system. This will be dependent on the material characteristics of the responder, its structural form, and the mass and character of the contents within the response zone and their spatial distribution, and includes both contents and people.

Whereas the avoidance of continuous (resonant) vibration problems generally involves isolating the source of the excitation from the responder, or ensuring the excitation frequency is sufficiently separated from that of the responder, transient vibration problems need to be specifically assessed. Two design procedures have gained widespread acceptance within the international building assessment community and are presented within Sections 7.5.2 and 7.5.3 below.

The first method (identified in this report as the Allen-Murray method – refer Section 7.5.2) was developed by Dr D.E. Allen (of the National Research Council of Canada) and Professor T.J. Murray (of Virginia State Polytechnic and University) (Allen & Murray 1993). It was developed to apply to commercial style buildings (concrete or steel frames with reinforced concrete decks) which have a natural frequency less than 10 Hz. The acceptability criteria are based on occupancy acceptance studies undertaken in the USA and Canada.

The second method (identified in this report as the Ohlsson Method – refer Section 7.5.3) was developed in Sweden by Professor Sven Ohlsson of Chalmers University. It has subsequently been included in the European Timber Design Code, EC 5 (EC 1995). It is specifically related to domestic style joist floor systems. The acceptance criteria applies only to floors with a natural frequency greater than 8Hz. Research at BRANZ (Beattie 1998) has indicated that the acceptance limits proposed by Ohlsson need to be tightened for application to New Zealand floors.

A static deflection limit is given in Table 1 of this report as a lower bound trigger value. If this requirement is satisfied vibration problems are then considered unlikely and more detailed investigations, such as those listed above and expanded in Sections 7.5.2

and 7.5.3 below, can generally be avoided. If the trigger value in Table 1 is exceeded, a more rigorous analysis should be undertaken.

7.2 Acceptable Vibration Acceleration

The excitation level at which vibrations cause problems is highly dependent on the sensitivity of the person exposed to the vibrations and to their circumstances when the vibration occurs. Thus there is no clear distinction between acceptable and unacceptable levels of vibration since personal sensitivities and highly variable external environmental factors combine to produce, at best, a fuzzy boundary of 'acceptable' response.

In an attempt to rationalise these external factors, and to provide quantitative guidelines for design, BS-6472:1984 'Guide to Evaluation of Human Exposure to Vibration in Buildings' (BSI 1984) contains acceleration levels of 'equal annoyance' (the 'base curve'). Two such curves are published within that Standard, the first for acceleration in the foot-to-head direction and the second for the front-to-back direction. In both cases the acceptance level varies with the frequency of vibration. In design, amplification factors are applied to these base curves to establish the nominally acceptable levels of acceleration below which vibration problems should be avoided. These amplification factors consider various environmental factors which influence acceptable response levels including the following:

Surrounding environment	tranquil or active surroundings (e.g. home, office or gymnasium)
Frequency of vibration	higher frequency accelerations (<40 Hz) are noticed less
Duration of vibration	short duration vibrations with higher accelerations are more tolerable
Expectation	events which are forewarned are more acceptable (conversely the responder may become more aware of the event having had forewarning)
Timing of vibration	motion at night is more annoying than the same motion during the day.

ISO 10137 (ISO 1992) brings together the combination of much of this work. Included in Annex C of this Standard are the amplification factors associated with the above environmental considerations, and these are reproduced in Table 7. The wide range of multipliers used indicate the diversity of acceptable vibration responses for the various occupancies. These multipliers are applied to a base acceleration curve also published in ISO 10137. However, the use of root mean square acceleration (rms) within that Standard results in a curve which is somewhat difficult to apply to design situations. Allan & Murray [1993] have translated this base acceptable acceleration curve into peak acceleration as % gravity and applied appropriate amplification multipliers, the results of which are presented here in Figure 2. In so doing, Allan & Murray make the following comments:

"For offices, ISO recommends multipliers of 4 for continuous or intermittent vibrations and 60 to 128 for transient vibrations. Intermittent vibrations are defined as a string of vibrations such as those caused by a pile driver, whereas transient vibration is caused rarely, for example by blasting. Walking vibration is intermittent in nature but not as frequent and repetitive as that caused by a pile driver. It is therefore estimated that the multiplier for walking vibration is in the range of 5 to 8, which corresponds to a root mean square acceleration in the range of 0.25% to 0.4% g for the critical 4 to 8 hertz range shown in Figure 2 (of this paper). Based on an estimated ratio of peak to rms acceleration of approximately 1.7 for typical walking vibration, the annoyance criterion for peak acceleration is estimated to be in the range of 0.4 to 0.7 percent g. From experience [Allen & Rainer, 1976], a value of 0.5 percent g is recommended for the frequency range 4 to 8 hertz. For footbridges, ISO recommends a multiplier of 60 which, combined with an estimated ratio of peak to rms acceleration of 1.7, results in a criterion of approximately ten times the vibration limit for offices. People in shopping centres will accept something in between, depending on whether they are sitting or standing".

Table 7: Multiplication factors used in several countries to specify satisfactory magnitudes of floor vibration with respect to human response [ISO 10137]

Place	Time	Multiplying factor to base curve	
		Continuous vibration & intermittent vibration	Impulsive vibration with several occurrences per day
Critical work areas (eg some hospital operating theatres, some precision laboratories, etc.)	Day Night	1 1	1 1
Residential (eg flats, homes, hospitals, etc)	Day Night	2 to 4 1.4	30 to 90 1.4 to 20
Offices (eg schools, offices)	Any	4	60 to 128
Workshops	Any	8	90 to 128

These combine as indicated in Figure 2. These curves are for acceleration in the foot-to-head direction as this is the normal attitude relating to vertical vibrations, or to horizontal vibrations when the subject is reclining.

7.3 Fundamental Frequency of Building Components

7.3.1 Beams

There are several methods for determining the fundamental frequency of vibration, f_o, for structural elements. For beam elements, Steffens [1974] suggests that the fundamental frequency can be given by equation (3).

$$f_0 = \frac{C}{\sqrt{\Delta_s}} \qquad (3)$$

where

Δ_s = the maximum static deflection in millimetres under the total imposed load (mm)

C = the frequency coefficient given in Table 8.

For continuous beams, the complete mode shape, including the response of the element over adjacent spans, needs to be considered when determining the natural frequency. When load is applied to a continuous beam, the downward deformation within the given span is matched with an upwards deflection within adjacent spans. The inertia effects are likewise in an upwards direction. Thus a continuous span can be shown to have the same fundamental frequency as that of a simply supported span when spans are roughly equal.

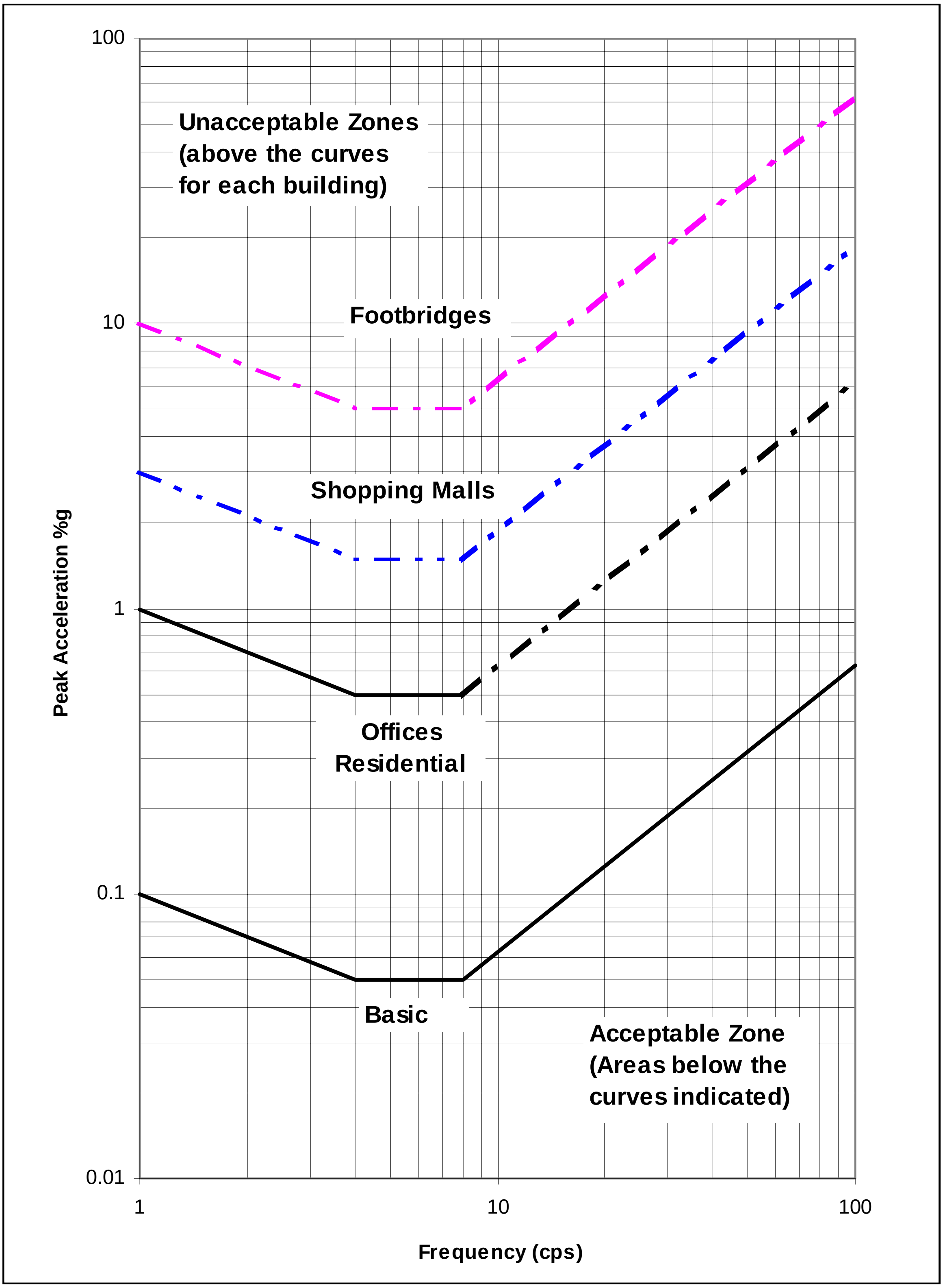

Figure 2: Acceleration limits to avoid walk induced vibration problems [from Allen & Murray 1993]

Table 8: Fundamental frequencies of beam elements [Steffens 1974]

Beam type	Load case	Static deflection (mm)	Frequency coefficient, C.	Self-weight correction factor
Cantilever	at end	$PL^3/3EI$	15.8	0.225
Cantilever	UDL	$PL^3/8EI$	19.6	1.0
Simply Supported	central	$PL^3/48EI$	15.8	0.5
Simply Supported	UDL	$PL^3/76.8EI$	17.7	1
Built in (Encastre)	central	$PL^3/192EI$	15.8	0.375
Built in (Encastre)	UDL	$PL^3/384EI$	17.7	1

Notes:
P = point load or total combined distributed load, in Newtons
L = the beam span, metres
E = modulus of elasticity of the material
I = second moment of area of the beam section

The self-weight correction factor is the proportion of the total beam weight, W_o, which must be added to the total applied load, P, to allow for the beam self-weight.

7.3.2 Two-way structural grids

Where a two-way structural grid system is present, then the fundamental frequency of the combined floor system should be assessed. This can most readily be achieved by using equation (4) [Steffens 1974]:

$$f_o = \frac{18}{\sqrt{\Delta_x + \Delta_y}} \quad (4)$$

Where Δ_x = the static deflection (mm) of the x direction spanning member (joist or beam) under the weight it is expected to support at the time of excitation (not the design load)

and Δ_y = the static deflection of the transverse y direction spanning member (girder) under the weight it supports (not the design load).

In all cases the composite action of the deck with the supporting element should be considered if such action is expected. It may significantly affect the dynamic response and load sharing between members. In determining the transformed sectional properties of such a composite system, Allen and Murray (1993) suggests that the dynamic stiffness of both the deck and the support beam should be considered. In the case of a concrete deck he suggests that the nominal modulus of elasticity be taken as 1.35 times that specified in Table 2 (and most structural material Standards).

For two-way concrete slab systems the American Concrete Institute [ACI 1974] suggests the following equations for determining the fundamental frequency for:

Hinged supports $$f_0 = \frac{18.7}{\sqrt{\Delta_x}} \quad (5)$$

Fixed supports $$f_0 = \frac{22.0}{\sqrt{\Delta_x}} \qquad (6)$$

For joisted floor systems, when considered as an anisotropic plate system, the fundamental frequency can be approximated by equation (7):

$$f_o = \frac{\pi}{2}\sqrt{\frac{K_x}{wL^4}}\sqrt{1+\left[2\left(\frac{L}{B}\right)^2+\left(\frac{L}{B}\right)^4\right]\frac{K_y}{K_x}} \qquad (7)$$

Where

f_o = the fundamental natural frequency (Hz)

$K_x = \frac{E_j I_j}{s}$ (i.e. the flexural stiffness of the joist member) with s being the joist spacing

$K_y = \frac{E_d t_d^3}{12}$ (i.e. the flexural stiffness of the deck applicable for joist systems only) with

E_d = the modulus of elasticity of the deck and

t = the deck thickness

E_f = modulus of elasticity of the flooring

L = span of the joist (in metres)

w = the mass of the floor per unit area including the average long-term live load (in kg/m^2)

B = the width of the floor (in metres)

For most ribbed floors K_x is much greater than K_y and equation (7) can be simplified to:

$$f_o = \frac{\pi}{2}\sqrt{\frac{K_x}{wL^4}} \qquad (8)$$

Equation (8) translates to:

$$f_o = \frac{\pi}{2}\sqrt{\frac{g}{q'}\left[\frac{E_x I_x}{sL^4}\right]} \qquad (9)$$

Where q' = dynamically active floor mass present (dead + average long-term live load)

$E_x I_x$ = stiffness modulus of the x direction spanning member (joist)

L = the member span in the x direction

s = the spacing between the x spanning members

g = gravity acceleration

This form is often more readily applied to beam and joist systems and can be shown to algebraically translate to equation (3) by substituting for Δ_s. **Always exercise great care in ensuring dimensional consistency is maintained.**

Example: Determine the fundamental frequency of vibration of a simple beam (using Table 8).

Consider a beam of 10 m span, second moment of area 10^9 mm^4, and weighing 150 kg/ m, which supports a central load of 100 kN. The material, steel, has a modulus of elasticity (E) of 210 GPa and the beam is simply supported.

When the self-weight of the beam is excluded, the fundamental frequency of the beam can be calculated from equation (3):

where $\Delta_x = \dfrac{\left(100x10^3\right)x\left(10x10^3\right)^3}{48x\left(210x10^3\right)x10^9} = 9.92mm$ and C = 15.8

then $f_0 = \dfrac{15.8}{\sqrt{9.92}} = 5.02Hz$

If the beam is unloaded, then the total applied weight, W_o= *10 x 150 (kg) = 1500 kg = 14,715 N*

$$\Delta_x = \frac{\left(14.7x10^3\right)x\left(10x10^3\right)^3}{76.8x\left(210x10^3\right)x10^9} = 0.91mm$$ and C = 17.7

then $f_0 = \dfrac{17.7}{\sqrt{0.91}} = 18.5Hz$

For the combined self-weight and central load, then P = 100 + 0.5 x14.7 = 107.36 kN

$$\Delta_x = \frac{107.36x10^3 x\left(10x10^3\right)^3}{48x210x10^3 x10^9} = 10.65mm$$ and C = 15.8

then $f_0 = \dfrac{15.8}{\sqrt{10.65}} = 4.84Hz$

7.4 Resonant Vibration Control

7.4.1 General

As discussed in Section 7.1 above, high levels of amplification occur when the response frequency of a supporting system approaches the frequency of excitation, particularly when the exciting mechanism continues over many cycles. Such resonant response may magnify the static displacement, velocity and/or acceleration of the support by up to 20 times that, which would result from the application of a static load of similar intensity.

The traditional construction techniques used prior to the 1970's resulted in relatively stiff buildings and building components which typically have a natural frequency of vibration greater than 20 Hz. Resonant response was generally not a problem since, apart from oscillating machinery (which could generally be isolated from the support system), there were few examples where this frequency of forcing function was encountered.

By contrast, modern forms of building, which tend to use more slender, high strength systems, also tend to have lower response periods and also less damping present. In such instances, Allen et al (1985) suggests that natural frequencies as low as 4 to 5 Hz are not uncommon. Such frequencies are particularly offensive to many people as can be seen from Figure 2, primarily because the internal body organs of humans also have a response frequency within this range. Furthermore, there is now a range of period excitation sources which can generate forcing functions with these characteristics. One, which is increasingly found to be responsible for causing floor vibration problems, relates to the rhythmic behaviour of coordinated crowds such as those present during dancing, gymnastics, concerts, spectator sports, and the like. Allen et al [1985] confirmed that crowds can be synchronised, by music or other means, to respond in unison at frequencies up to 6 Hz. Beyond this they become discoordinated and a random forcing function results. Thus, there are now several instances where the building response is within a range which is highly aggravating to people, and there are also several situations where the forcing function can be expected to coincide with this response frequency. Thus within Clause 2.5.2.3 of NZS 4203 [SNZ 1992], there are provisions for checking the dynamic response of systems which are required to support crowds of people where they are likely to behave in a harmonically excited manner. Such floors are required to be checked for resonance. A design procedure for such a check, which has been developed to avoid resonant response thereby reducing the potential for related structural damage, is detailed in Section 7.4.2. It does not necessarily avoid transient vibration problems and further checks using the procedures outlined in Section 7.5 may be required.

7.4.2 A design procedure to avoid resonance

The following procedure recommended by Allen et al [1985] can be applied to identify floor systems that are likely to resonate when subjected to cyclic excitations. To avoid this phenomenon, the component may need to be stiffened to increase its natural frequency beyond the range of the exciting forces. The procedure is specifically for buildings that are subjected to periodic excitation generated by human activities, but, provided the dynamic characteristics of the excitation function are known, it can also be applied to other forms of excitation.

The following design procedure may be used to check a floor system for resonant response:

1. Assess the loading of the area, recognising the type of activity, and hence the density of occupancy and the probable distributed weight of participants, Wp (refer Table 10).

2. Select an appropriate forcing frequency, f, and dynamic load factor, α, which is applied to Wp to determine the dynamic load αWp (refer Table 10). For jumping exercises, the frequency for both the first, second and third harmonics (2.75 , 5.5, and 8.25 Hz) should be checked.

3. Select an acceptable limiting acceleration, a_o, at the centre of the floor (refer Table 9 and Figure 2 for guidance).

4. Estimate the total floor load, Wt (i.e. dead load + αWp).

5. Determine the fundamental frequency, f_o, for the floor structure (include self-weight plus actual applied live load) using the appropriate equation from Section 7.3.

6. Check that the following condition is satisfied:

$$f_0 > f\sqrt{1+\frac{K}{a_0 / g}\left(\frac{\alpha W_p}{W_t}\right)} \qquad (10)$$

where f_o = the fundamental frequency of the supporting element (Hz) (refer Section 7.3)

f = the forcing excitation frequency (Hz)

Wp = weight of participants (kPa) (refer Table 11)

α = the dynamic load factor

Wt = the total load (kPa)

a_o/g = limit of 'acceptable' acceleration, ratio of g, (refer Table 9 and Table 11)

K = 1.3 for most (fixed & simply supported) beams

= 1.5 for cantilevers and two-way slabs

= 2.0 for rhythmic jumping exercises.

If the criterion is satisfied, the level of acceleration will generally be acceptable for the occupancy and activity being considered. Where the criterion is not met, remedial measures which may be considered include stiffening the structure, relocating the activity, controlling the activity (i.e. f and Wp) or accepting a higher level of acceleration and possible annoyance problems.

The basis for the acceptable acceleration limits, a_o, introduced in stage 6 of the above evaluation, are outlined in Table 9 and Table 11. These were derived from measurements made on unacceptable floors experienced in the field in Canada [Allen, 1990; Allen, 1990(a)] and Europe [Bachmann & Ammann, 1987].

7.4.3 Estimation of dynamic load parameters

Allen et al [1985] outlined a methodology whereby the dynamic load parameters for various rhythmic human activities could be ascertained. The first step is to determine the frequency of excitation to which the floor may be subjected. The excitation frequencies and distribution of live loads for many such activities are given in Table 10.

The dynamic load factor, α, is used to determine the dynamic load as a proportion of the total active load. Values are given for some problem-causing activities (see Table 10). These are based on a minimum of 20 people and may be higher for fewer, highly synchronised events. A probable distributed weight of participants to be used in conjunction with the dynamic load factor is given.

Table 9: Acceptable acceleration levels for vibrations due to rhythmic activities

Occupancy Affected by the Vibration	Acceleration Limit (% gravity)
Office and residential	0.4 - 0.7
Dining and weightlifting	1.5 - 2.5
Rhythmic activities only	4 - 7

Notes: From Table A3 of the Supplement to the Canadian Building Code (1990). The broad range of acceptable accelerations reflects the complex nature of this assessment which includes both the activity of the exciting source and that of the receiver. Duration and remoteness of the receiver from the source are other factors to consider. The lower value is recommended for design.

Table 10: Estimated dynamic loading during rhythmic events

(From Table A-2 from Appendix A of Chapter 4 of the Supplement to the National Building Code of Canada 1990)

Activity	Forcing frequency f,(Hz)	Weight of Participants[(1)] Wp, (kPa)	Load Factor[(2)] α	Dynamic Load αWp,(kPa)
Walking	1.5 to 2.5	0.8 (l m^2/person)	0.6	0.48
Jogging	2.5 to 3	0.6 (1.2 m^2 /person)	0.4	0.24
Dancing	1.5 to 3	0.6 (2.5 m^2 /couple)	0.5	0.3
Lively concerts or sports events	1.5 to 3	1.5 (0.5 m^2/person)	0.25	0.4
Jumping Exercises				
First Harmonic	2 to 2.75	0.2 (3.5 m^2/person)	1.5	0.3
Second Harmonic	4 to 5.5	0.2 (3.5 m^2/person)	0.6	0.12
Third Harmonic	6 to 8.5	0.2 (3.5 m^2/person)	0.1	0.02

Notes (1) The density of participants represents maxima for commonly encountered conditions. For special events the density of participants can be greater.

(2) Values of α based on commonly encountered events with a minimum of 20 participants. Values of α should be increased for well coordinated events with fewer than 20 participants.

Table 11: Application of vibration criteria for human reaction to different activities and floor constructions

Type of floor	Forcing Frequency f,(Hz)	Effective Weight of Participants Wp,(kPa)	Total Weight Wt, (kPa)	Minimum Fundamental Frequency f_o, Hz
Dancing and dining, a_o = 2% g				
concrete (5kPa)	3	0.6	5.6	6.4
steel joist (2.5 kPa)	3	0.6	3.1	8.1
timber (0.7 kPa)	3	0.6	1.3	12.0
Lively concert or sports event, a_o = 5% g				
concrete (5kPa)	3	1.5	6.5	4.8
steel joist (2.5 kPa)	3	1.5	4.0	5.7
timber (0.7 kPa)	3	1.5	2.2	7.2
Jumping exercises only, a_o = 6% g				
concrete (5kPa)	8.25 [(1)]	0.2	5.2	8.8
steel joist (2.5 kPa)	8.25 [(1)]	0.2	2.7	9.2
timber (0.7 kPa)	5.5 [(1)]	0.2	0.9	12.8
Jumping exercises with weight training, a_o = 2% g				
concrete (5kPa)	8.25 [(1)]	0.12	5.12	9.2
steel joist (2.5 kPa)	5.5 [(1)]	0.12	2.62	10.6
timber (0.7 kPa)	5.5 [(1)]	0.12	0.82	17.2

Notes: Reproduced from Table A-4 of the Supplement to the National Building Code of Canada which was compiled using Equation 10 from Section 7.4.2 using K=1.3 for beam elements, except for jumping activities where K=2.0.

[(1)] Equation 10 is applied to each harmonic (ie. f=2.75 Hz, 5.5 Hz and 8.25 Hz) and the governing harmonic is used in Table 11.

7.5 Transient (Walking) Floor Vibration Control

7.5.1 General

One of the most common floor vibration problems relates to floor liveliness where the movement of one person is felt by another and found disturbing. Considerable research has been undertaken into devising design assessment methods which could be used to predict and control this phenomenon during the design of floor systems. Two of the more widely accepted methods are presented here. The first, the Allen-Murray method (Allen & Murray, 1993), is a joint US/Canadian approach with the acceptance levels being derived from measurements of problem floors within these communities. It specifically relates to floors with a fundamental frequency of less than 9 Hz where constant accelerations are the appropriate control parameter Figure 2). It has been specifically derived from the consideration of a two-way steel frame support with a concrete deck. It can be applied to a concrete frame with a concrete deck but it should not be applied to strongly orthotropic floors such as joist floors. It may be extended beyond 9 Hz when footstep impulse response becomes important, as long as it is used in conjunction with the stiffness criterion of 1.5 kN/mm. However, velocity is the preferred performance indicator above 9 Hz. The criterion is to appear in a guideline on floor vibrations currently being prepared for the American and Canadian Institutes of Steel Construction.

In contrast, the Ohlsson method (refer Section 7.5.3) applies to highly orthotropic floors (an abbreviation of orthogonally anisotropic – i.e. having large stiffness variation in two perpendicular directions) with a fundamental frequency greater than 8 Hz (Ohlsson 1988). In this case the acceptance criterion limits the maximum velocity resulting from a unit impact to a specific value. The floor system is assumed to be an orthotropic slab with a contribution from all frequencies less than 40 Hz being considered. The acceptance criteria relate specifically to domestic construction and were derived from studies in Sweden. They form the basis for the floor vibration criteria stipulated within Eurocode 5 'Timber Structures'. However, the acceptance criteria have been relaxed to reflect what is considered to be acceptable performance within New Zealand and Australian metal framed houses. It forms the basis for the dynamic floor response assessment section of AS 3623:1992 'Domestic Metal Framing' (SAA 1992). Research undertaken at BRANZ by Beattie [1998] has lead to the proposal that the Ohlsson criteria is too liberal for timber floor joists used in New Zealand houses and that these criteria should be tightened. This is to be coupled with an increase in the level of damping assumed from 1% to 2% which is consistent with values measured during laboratory trials undertaken at BRANZ and reported by Beattie.

7.5.2 Allen-Murray design criterion for vibrations due to walking (floors f_o<9 Hz)

Floor liveliness problems experienced by long-span floors are commonly able to be attributed to quasi-resonant effects initiated by the impact of human footfalls. Walking forces have been found to produce up to three harmonic frequencies below 9 Hz viz approximately 2, 4 and 6 Hz [Allen & Murray, 1993]. The closeness of these frequencies results in an ideal scenario for resonance for floor systems with a fundamental frequency of less than 9 Hz. For those floors of this nature which have a fundamental frequency greater than 9 Hz, harmonic resonance does not occur and static deflection control measures should be sufficient to ensure adequate performance (refer to item S15 of Table 1). The Allen-Murray approach is inappropriate for floors with a natural frequency of greater than 9 Hz and should not be used, unless in conjunction with the static deflection controls.

Three other factors which also influence floor vibration and thus the potential for vibration problems are:

- human reaction to floor vibration (which depends on the use and occupancy of the floor)
- the proximity of the walker to the middle of the floor panel, and
- the distance between the walker and the annoyed respondent.

Within this discussion, the terms 'joist' and 'beam' are used to describe the secondary support member (x spanning), and 'girder' to describe the primary support (i.e. that which supports and spans perpendicular to the joist – y spanning).

The essence of applying this procedure is as follows:

1) Ascertain the occupancy classification of the floor in question and assign an appropriate constant acceleration limit, K, to that occupancy. Refer Table 12 for guidance.

2) Ascertain the effective dynamic panel weight, W, for both the x-spanning (joist) floor panel and for the y-spanning (girder) panel (where the response of the girder is dynamically significant). In each case, the effective panel weight can be derived for each direction from equation (11),

$$W_i = w\, B_i\, L_i \qquad (11)$$

where W_i = dynamically active weight of floor per unit area in direction i (kN)

w = the average long-term live load plus the dead load ($G + 0.5\ \Psi_L\ Q$) (kPa)

B_i = effective panel width for member spanning in direction i (m) *{Note: this width is perpendicular to the member span}*

L_i = member span in direction i (m)

i = x or y for joist or girder respectively

Where the joists are continuous over their support, and the adjacent span is not less than 0.7 $L_{j,}$ then the joist panel weight can be increased by 50% to reflect the engagement of adjacent floor panels in the fundamental mode of vibration.

3) Determine the fundamental frequency, f_o, of the floor system either by reference to equations (3) or (4) or other means. Consider both two-way action (where both the floor joist and supporting girder are considered flexible) and composite beam/deck action. Generally the floor self-weight and one half the long-term live load (for non-storage occupancies) will be considered to be present for determining the mid-span elemental deflection and thus fundamental frequency.

4) The effective panel width, B_i, is the width perpendicular to the member over which the floor may be considered to be dynamically active. It is determined for members spanning in direction x as shown in equation (12) below (panel width in y direction is similar).

$$B_x = C\left(\frac{D_y}{D_x}\right)^{\frac{1}{4}} L_x \qquad (12)$$

Where D_y = flexural rigidity perpendicular to the member (= $E_y\ I_y/L_y^4$)

D_x = flexural rigidity in the member span direction (= $E_x\ I_x/L_x^4$)

L_x = member span (m)

C = 2.0 for joists or beams in most locations except when they are beside an internal opening (e.g. mezzanine or stair well)

= 1.0 for joists or beams beside an internal opening

= 1.4 for girders supporting joists from the top flange only (i.e. joists simply supported)

= 1.7 for girders supporting beams or joists from both upper and lower flange

The form of equation (12) is based on orthotropic plate action.

The panel width for the member spanning in a transverse direction, determined from equation (12), need not be less than the tributary panel width supported by the girder, nor greater than 2/3 the total floor width perpendicular to the girder.

Since deflections are small, being within the serviceability limit state, it is appropriate to use the (average) uncracked thickness of the (ribbed) floor deck systems, and a dynamic modulus of elasticity (=1.35 E) when assessing the flexural rigidity of the deck. The effective slab width is considered to be equal to the joist spacing but not more than 0.4 times the member span or 8 times the slab thickness. Where the member forms an edge beam, half this value plus the projected width is appropriate.

5) Ascertain the equivalent panel mass of the combined system by using the approximate interaction formula given in equation (13), where W_x, $\Delta_{x,}$, W_y and Δ_y are the panel weight and static deflection related to the x & y-spanning members respectively.

$$W = \frac{\Delta_x}{\Delta_x + \Delta_y} W_x + \frac{\Delta_y}{\Delta_x + \Delta_y} W_y \qquad (13)$$

If the y-span, L_y, is less than the x-panel width, B_x, then the combined mode is restricted with the system being effectively stiffened. This can be accounted for by reducing the static deflection of the y member, Δ_y in accordance with equation (14).

$$\Delta_y' = \frac{L_y}{B_x}\left(\Delta_y\right) \qquad (14)$$

6) The damping ratio, ξ, needs to be assessed for the particular form of construction being considered. Table 12 provides damping ratios for typical concrete deck and steel floor systems.

7) The acceptance criterion for minimising walking vibration problems applies to all floors with a fundamental frequency of less than 9 Hz. The criterion is based upon equation (15). Values of K are also given in Table 12 for different occupancies. These have been plotted in Figure 3:

$$^{-0.35f_0} \qquad (15)$$

Equation (15) can be rewritten to make f_0 the subject as indicated in equation (16)

$$f_0 \geq 2.86 \log_e \left(\frac{K}{W\zeta} \right) \qquad (16)$$

Table 12: Values of dynamic floor response factor, K, and damping ration, ξ.

Floors	K (kN)	ξ
Office residence and churches	58	0.03*
Shopping Malls	20	0.02
Footbridges	8	0.01

* ξ = 0.05 for floors with floor to ceiling partitions

= 0.02 for floors with few non-structural components (such as ceilings, ducts partitions etc) as can occur in churches.

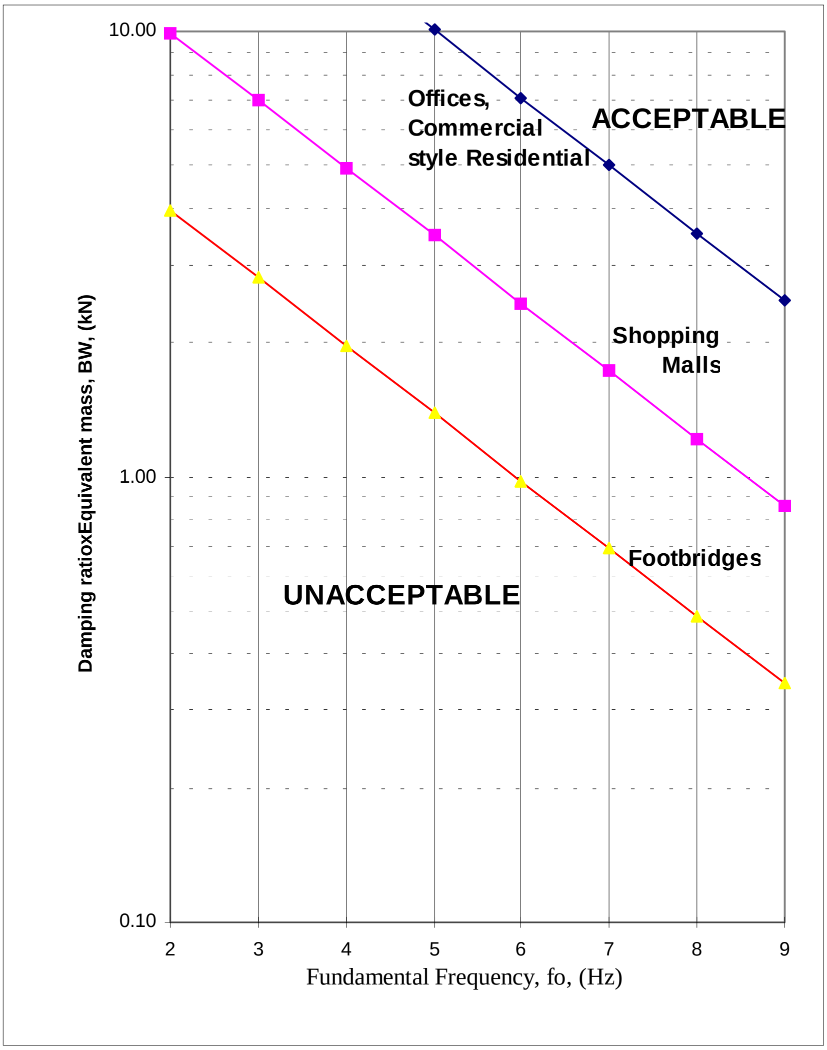

Figure 3: Acceptability criteria for walking induced vibrations of floors (with fo< 9 Hz) (after Allen & Murray 1993)

Example 1 (below) has been prepared to show how the above criteria can be applied. The example is intentionally complex in order that all aspects of the procedure are displayed. A ribbed reinforced concrete floor spans over 310UB46 joists (spacing 2.5 m; spanning 8.5 m). The joists are in turn supported by 530UB92 girders (spacing 8.5 m; spanning 10 m). Both joists and girders are in full contact with the deck and full composite action is available between the deck and each support member. The particular panel being considered is an internal panel with an office occupancy.

Example 1: Dynamic Floor Response - when frequency F_0 <9 Hz

Example: Dynamic response of a ribbed concrete floor in composite action with a 310UB46 steel joists and 530UB92 steel girders

Longitudinal Member Properties

Joist Properties	Material	=	Steel	Member Depth	=	310	
	Area mm2	=	5890 mm	Ej	=	210.0 Gpa	
	I value mm4	=	99.5E+6 mm4	γj	=	7800 kg/m3	
	Aj	=	5890 mm2				
Joist member = 310UB46	Ij	=	99.5E+6 mm4	Mass/m run	=	46 kg/m	
Span	Lj	=	8.50 m	Percentage Flange Action			
Spacing	Sj	=	2.50 m	Deck to Joist		100	ie shear studs included

Transverse Elemental Properties

Deck Properties	Material	=	Concrete				
				Ec	=	24.0 Gpa	
Deck Thickness	t_{slab}	=	90 mm	Dynamic Stiffness		32.4 Gpa	
Rib Depth	d_{rib}	=	40 mm2	γc	=	2000 kg/m3	= 20 kg/m
	Avg Depth	=	110 mm				
Joist Modular Ratio (Dynamic)= Ej/1.35Ec		=	6.48				

Girder or Bearer Properties	Material	=	Steel	Member Depth	=	533	
	Area mm2	=	11800 mm	Eg	=	210.0 Gpa	
	I value mm4	=	554.0E+6 mm4	γg	=	7800 kg/m3	
Girder Member = 530UB92	Ag	=	11800 mm2				
	Ig	=	554.0E+6 mm4				
Span	Lg	=	10.00 m	Mass/m run	=	92 kg/m	
Spacing	Sg	=	8.50 m	Percentage Flange Action			
Girder Modular Ratio (Dynamic)= Eg/1.35Ec		=	6.48	Deck to Girder		100	ie shear studs included
Occupancy Class	Office			g	=	9.81 m/sec2	

Self Weight

Decking Weight	19.62 x	110	x	0.001	=	2.16 kPa
Average Long Term Live Load	0.5 x	0.4 x		3.00 kPa	=	0.60 kPa
Allowance for Mechanical & Ceilings					=	0.20 kPa
Weight of Joist (as UDL)	46 kg/m x	0.00981		2.50 m	=	0.18 kPa
		Dynamically Active Joist UDL			=	3.14 kPa
Weight of girder (as UDL)	92 kg/m x	0.00981/		8.50 m	=	0.11 kPa
		Dynamically Active Girder UDL				3.24 kPa

Joist Sectional Properties

Effective Transformed Section Width Minimum of 0.25 Lj (Span) or Spacing, Sj or 2 x 8 t_{slab} (from NZS 3101)

2.13 m 2.50 m 1.76 m = 1.76 m

	Eff Area	y mm	(Eff A)*y mm3	Dist to Centroid	Ay2	2nd M of A
Deck	29870	55	1642834	38	42.9E+6 mm4	30.1E+6 mm
Joist	5890	285	1678650	-192	217.4E+6 mm4	99.5E+6 mm
	35760 mm2		3321484 mm3		260.3E+6 mm4 +	129.6E+6 mm
	Centroid depth= 92.88					389.9E+6 mm

Girder/Bearer Sectional Properties

Transformed Girder Section Properties

Effective Transformed Section Width 0.25 Lg (Span) or Spacing Sg or 2 x 8 t_{slab} (from NZS 3101)

Minimum of 2.50 m 8.50 m 1.76 m = 1.76 m

	Eff Area	y mm	(Eff A)*y mm3	Dist to Centroid	Ay2	2nd M of A
Deck	29870	45	1344137	100	295.9E+6 mm4	20.2E+6 mm
Girder	11800	397	4678700	-252	749.1E+6 mm4	554.0E+6 mm
	41670 mm2		6022837 mm3		1.0E+9 mm4	574.2E+6 mm
	Centroid depth= 144.5					1.6E+9 mm

Joist Serviceability Checks

UDL Load Combinations G+Q	2.54 kPa +	3.00 kPa x	0.70	=	4.64 m	
Static Deflection (Joist) =			9.6 mm	=	Span / 883	OK
Static Deflection (Girder) =			6.0 mm	=	Span / 414	OK
1 kN Point load deflection =			0.2 mm	=		OK for deflection

Dynamic Properties - Joist Direction

Effective Joist Panel Width, $B_j = 2 * \left(\frac{D_s}{D_j}\right)^{\frac{1}{4}} * L_j$

Flexural Stiffness in Slab direction Ds =	1000 mm	x	1.3E+6 x	32.40	/		12)=		3.6E+9 kPa/m
Flexural Stiffness in Joist Direction Dj=	389.9E+6 x		210.0 Gpa	/	2.50 m			=	32.7E+9 kPa/m
Bj =	2	x(	3.6E+9	/	32.7E+9	$)^{(0.25)}*$	8.50 m	=	9.78 m

Check panel width < 2/3 of the total floor width perpendicular to the joist span

Actual Floor width (interior bay) =	3	x	10.00 m		=	30.00 m		
2/3 floor width =	2	x	30.00 m /	3	=	20.00 m		
			Thus Bj assessed < 2/3 floor width			**So Bj**	=	9.78 m

Mass of Joist Panel Weight (including 50% increase to allow for joist continuity)

Wj =	1.5	x	3.14 kPa x	9.78 m	x	8.50 m	=	392 kN

Joist response

Midspan Joist Deflection = $\Delta_{sj} = \frac{5 w_j L_j^4}{384 E_j I_j}$ = 6.5 mm = Span / 1305

Joist Natural Frequency $f_0 = \frac{C}{\sqrt{\Delta_s}} = \frac{17.7}{\sqrt{\Delta_s}}$ = **6.94 Hz**

Dynamic Properties of Girder/Bearer

Effective Girder Panel Width, $Bg = 1.8 * \left(\frac{Dj}{Dg}\right)^{\frac{1}{4}} * Lg$

Flexural Stiffness in Joist direction Dj =								=	32.7E+9 kPa/m
Flexural Stiffness in Girder Direction Dg=	1.6E+9 x		210.0 Gpa	/	8.50 m			=	40.0E+9 kPa/m
Bg =	1.8	x(	32.7E+9	/	40.0E+9	$)^{(0.25)}*$	10.00 m	=	17.12 m

Check panel width < 2/3 of the total floor width perpendicular to the girder span

Actual Floor width (interior bay) =	3	x	8.50 m		=	25.50 m		
2/3 floor width =	2	x	25.50 m /	3	=	17.00 m		
			Thus Bg assessed > 2/3 floor width			**So Bg**	=	17.00 m

Girder Panel Weight (No continuity)

Wg =	1	x	3.24 kPa x	17.00 m	x	10.00 m	=	552 kN

Girder Response

Equiv. Static Deflection for Girder Assume Simple support $\Delta_{sg} = \frac{5 w_g L_g^4}{384 E_g I_g}$ = 10.6 mm = Span / 947

Girder Fundamental Frequency $f_0 = \frac{C}{\sqrt{\Delta_s}} = \frac{17.7}{\sqrt{\Delta_{sg}}}$ = 5.45 Hz

Check that joist panel width < girder span & reduce girder deflection if necessary

Girder span< Joist Panel Width, so 9.78 m < 10.00 m = TRUE So unmodified Dsg = **10.6 mm**

Combined Mode Dynamic Properties

Fundamental Frequency $f_0 = \frac{17.7}{\sqrt{\Delta_{sj} + \Delta_{sg}}}$ = **4.28 Hz < 9.00 Hz**

Thus Within the Theory Range

The combined panel weight $Wt = \frac{\Delta j}{\Delta j + \Delta g} Wj + \frac{\Delta g}{\Delta j + \Delta g} Wg$

Wt =	0.38	x	392 kN	+	0.62	+	552 kN	= 491 kN

Evaluation

For Office Occupancy without full height partitions, Damping ratio suggested = 0.03

Then ξ W = 0.03 x 491 kN = 15 kN

Require $f_0 > 2.86 \ln(K/\xi W)$ where K = 58

Then f_0 Should Be Greater than	2.86	x	log	(	58	/	15 kN	)=	3.92 Hz
							And	f_0 =	4.28 Hz

Conclusion Floor probably acceptable

7.5.3 'Modified' Ohlsson design criterion for vibrations due to walking (floors f_o>8 Hz)

This criterion is applicable only to joist floor systems and only those which have a fundamental frequency greater than 8 Hz. The criterion was developed by Ohlsson (1988). The floor systems are considered as orthotropic. The control criterion is based on limiting the unit impulse velocity to below a given level, that level being a log function of the fundamental frequency of the floor and the degree of damping present. The original criteria published by Ohlsson are shown in Figure 4 as the upper and lower bound of the central zone, indicated as 'Marginal'. A third line has been added to reflect the proposed acceptance criterion for domestic floor systems as given in equation (17) below. This has been assessed from floor joist span tables used in light timber framed flooring codes within New Zealand and Australia (Carson 1994) which have generally been found to perform satisfactorily.

$$Log_{10}V_{\max} < 1.2 + 2\sigma_o \qquad (17)$$

Research undertaken at BRANZ [Beattie 1998] attempted to use this criteria to separate acceptable from unacceptable floors built according to New Zealand construction practice. It became apparent that several floors, although passing as 'acceptable' using Ohlsson's criteria, were clearly unacceptable in practice. This resulted in a change to the acceptance criteria to that indicated in Figure 4 which is to be used in conjunction with the greater system damping ratio of 2%. Thus the acceptable performance criteria for floors covered by the section is indicated in equation (18). When this inequality is met the performance is expected to perform satisfactorily. The lesser bound conditions can be used when some springiness is acceptable and the criteria can be relaxed.

$$Log_{10}V_{\max} < 0.3 + 2\sigma_o \qquad (18)$$

Where	$V_{\max}$	=	maximum unit impulse velocity
and	σ_o	=	damping coefficient = $f_o\ \xi$
	f_o	=	fundamental frequency of vibration
	ξ	=	modal damping ratio

Equation (18) can be rewritten to make the maximum unit impulse velocity the subject as indicated in equation (19):

$$V_{\max} < 2x100^{\sigma o} \qquad (19)$$

The impulse velocity, V_{max}, can be approximated by:

$$V_{\max} = 4\left[\frac{0.4 + 0.6N_{40}}{wBL + 200}\right]$$

$$N_{40} = \frac{B}{L}\sqrt{\sqrt{\left(\frac{r + f^2 - 1}{r}\right)} - 1}$$

where N_{40} = the number of modes with natural frequency less than 40 Hz

r = the ratio of the floor stiffness in the two directions ($=K_y/K_x$)

f = the frequency ratio ($40 / f_o$.)

For floors with K_x much greater than K_y, then N_{40} can be further approximated by:

$$N_{40} \approx \frac{B}{L}\left(\frac{f^2-1}{r}\right)^{1/4}$$

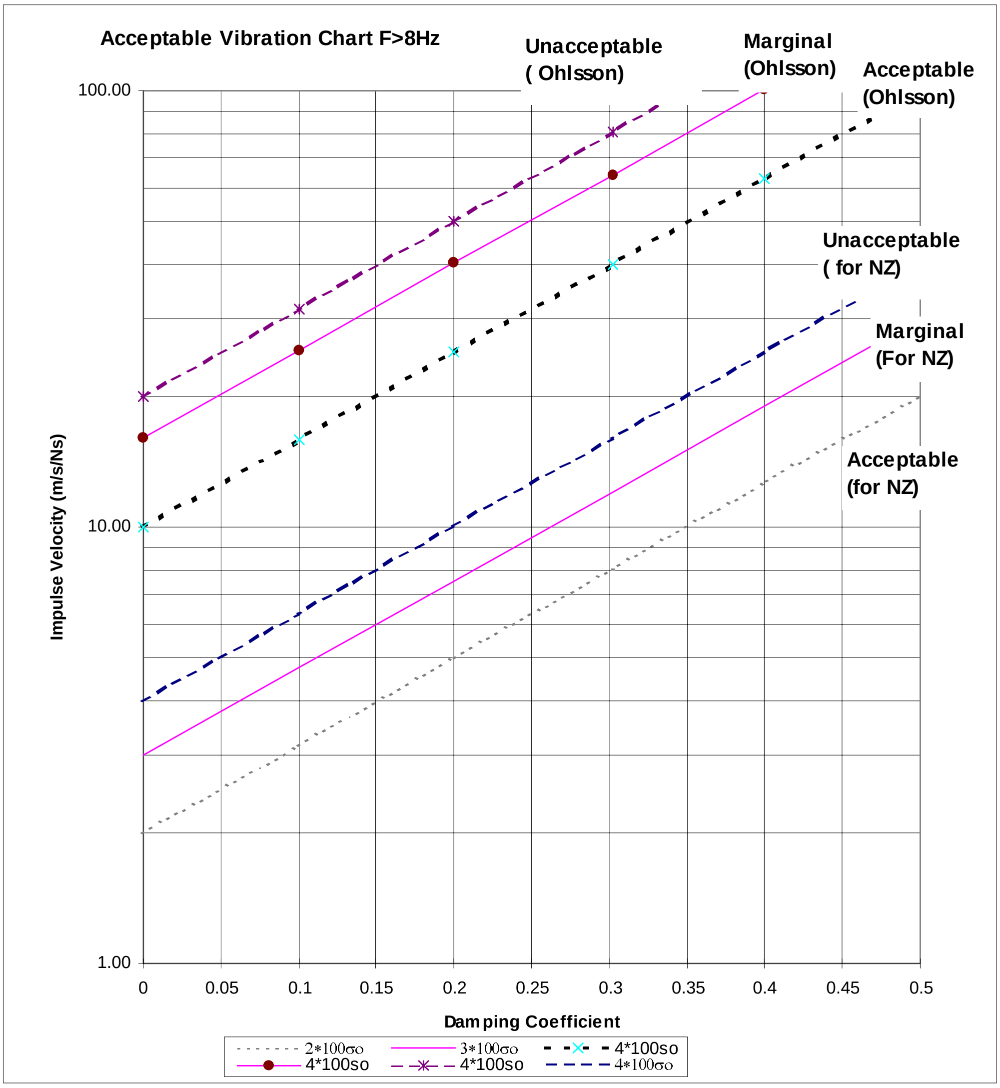

Figure 4: Acceptable unit impulse velocities for floors with F_o> 8 Hz (after Ohlsson 1988)

Example 2 (below) shows the calculations required for such an evaluation. The procedure has a number of input values which must be estimated for the conditions present, such as the continuity of the floor decking and the effectiveness of blocking and ceiling lining. By adjusting these values for each laboratory test configuration, Beattie [1998] matched the first mode natural frequency and number of modes below 40 Hz in the experimental testing with that predicted by the Ohlsson procedure. He found that the following conservative generalisations could be made:

1. When installed in conjunction with ceiling battens and plaster based ceiling lining, the effectiveness of one or two rows of battens and plaster ceiling linings can be assumed to be at least 70%.

2. The effectiveness of ceiling battens and plaster ceiling linings can be assumed to be approximately 45%, provided there is a least one row of blocking present.

3. One row of blocking and no ceiling can be assumed to be approximately 40% effective.

4. Two rows of blocking can be assumed to be at least 59% effective when no ceiling is present.

5. The percentage continuity provided by the deck ranges from 20% at 3.2 m span to 60% at 4.8 m span.

6. The addition of adhesive between the deck and the joists does not enhance the degree of continuity provided by the deck over that established for nails alone.

Note that the solution is highly sensitive to the transverse stiffness and that the most effective way of improving problem floors is to increase the transverse stiffness, thereby activating a greater width of floor and initiating more load sharing between joists. This is best achieved by the combined use of transverse blocking and ceiling lining. The addition of blocking alone is markedly less effective.

Example 2 Dynamic Floor response - Domestic Floors where F_0>8 Hz

Primary Joist (X) Properties

Primary Joist (X) Properties	Material	=	Timber	Grade	=	No 1	
Joist member = 191 x 46	Depth	=	190 mm	Ej	=	8.0 Gpa	
	Width	=	45 mm	Dynamic Stiffness		8.0 Gpa	
	Aj	=	8550 mm2	γj	=	450 kg/m3	= 0.04 kN/m
	Ij	=	25.7E+6 mm4	K2 =		2	
Primary Joist Span	Lj	=	3.50 m				
Joist Spacing	Sj	=	0.40 m				

Properties of Transverse (Y) Elements

Transverse Deck Span between support walls	Ly =		8.00 m	(Note Should be > joist span)			
Deck Properties	Material	=	PBd	% Continuity	Deck to Joist		30
	(Timber, Ply, PBd,Concrete)			Ed	=	4 Gpa	
Deck Thickness	t_{slab}	=	20 mm	Eff. Dynamic Stiffness		4.0 Gpa	
Rib Depth	d_{rib}	=	0 mm	Dynamic Stiffness ratio		0.50	
	Avg Depth	=	20 mm	γd	=	690 kg/m3	= 7 kN/m3
Transverse Blocking	material or "NA" if Not present	=	timber	(Timber,Al,Steel,NA)			
	Depth	=	190 mm OK	Grade	=	No 1	
	Width	=	45 mm	Ebk	=	8.0 Gpa	
	Aybk	=	8550 mm2	Dynamic Stiffness		8.0 Gpa	
	Iybk	=	25.7E+6 mm4	γbk	=	450 kg/m3	= 0.04 kN/m
Span	Lybk	=	0.40 m	% effective =	=	70 %	
Number of Blocks within Joist Span	Nbk	=	2 No.				
Ceiling Battens	material or "NA" if No response	=	timber	(Timber,Al,Steel,NA)			
	Depth	=	35 mm	Timber grade =		f8	
	Width	=	75 mm	Ecb	=	8.0 Gpa	
	Acb	=	2625 mm2	Dynamic Stiffness		8.0 Gpa	
	Icb	=	268.0E+3 mm4	γcb	=	450 kg/m3	= 0.01 kN/m
Span	Lcb	=	0.40 m				
Spacing	Scb	=	0.60 m				
Ceiling	Material	=	Gib	% Continuity	Joist to Ceiling		45
	(Timber, Ply, Gib, Plaster, NA)			Ec	=	3 Gpa	
Thickness	$t_{ceiling}$	=	10 mm	Eff Dynamic Stiffness		3.0 Gpa	
				Effective Dynamic Stiffness ratio		0.38	
				γc	=	750 kg/m3	= 7 kN/m3

Deck Plate Dynamic Geometry

Width, L	=	3.50 m	Note Always assumes joist span min direction so B/L>1.0	
Breadth B	=	8.00 m	Then B/L =	2.29
Occupancy Class	Residential		g =	9.81 m/sec2

Self Weight

Decking Weight	7 kN/m3	x	20 mm	x 0.001	=	0.14 kPa
Ceiling	10 mm	x	7 kN/m3		=	0.07 kPa
Battens	0.01 kN/m	/	5 kg/m3		=	0.00 kPa
Blocking	0.04 kN/m	/	1.75 m		=	0.02 kPa
Allowance for Mechanical & Ceilings					=	0.10 kPa
Weight of Joist (as UDL)	0.04 kN/m	/	0.40 m		=	0.09 kPa
Average Long-Term Live Load	0.5	x	0.4 x	1.50 kPa	=	0.30 kPa
				Dynamically Active Joist UDL	=	0.72 kPa

Sectional Properties Along Joist (X) direction

Effective Transformed Section Width	0.25 Lj (Span) or	Spacing, Lj or	2 x 8 t		
Deck Minimum of values or zero if ineffective	0.88 m	0.40 m	0.32 m	=	0.10 m
Ceiling Minimum of values or zero if ineffective	0.88 m	0.40 m	0.15 m	=	0.07 m

	Eff Area	y	Eff A * y mm3	Dist to NA		Ay2	I
Deck	960 mm2	10	9600 mm3	94 mm	x	8.6E+6 mm4	8.0E+3 mm
Joist	8550 mm2	115	983250 mm3	-11 mm	x	960.6E+3 mm4	25.7E+6 mm
Ceiling	244 mm2	250	60858 mm3	-145 mm	x	5.1E+6 mm4	1.3E+6 mm
Sum =	9510 mm2		992850 mm3			14.7E+6 mm4	25.7E+6 mm
			Then N/A depth =	104 mm		Gross 2nd M of A =	**40.4E+6 mm4**

Static Serviceability Check

G= 0.42 kPa Q= 1.50 kPa Ψ_s 0.7 Ψ_s 0.40

		Long Term		Short Term		Total
Static Long Term Joist Deflection	$\Delta_{s,UDL} = \frac{5wL^3}{384K_2EI}$	1.2 mm	+	1.1 mm	=	2.3 mm
						Span / 1505
Joist Deflection under 1 kN point load	$\Delta_{sp} = \frac{PL^3}{48EI}$	2.8 mm		**Dynamic Check Required**		

Orthogonal Stiffness Assessment

Longitudinal Flexural Stiffness per m of X, Dx

40.4E+6 x 8.0 Gpa / 0.40 m = 807.9E+3 Nm2/m

Transverse Flexural Stiffness per meter of Y, Dy

Deck	1.00 m	x	8.0E+3	x	4.0 Gpa	/	12	)=	2.7E+3 Nm2/m
Blocking	70 %	x	25.7E+6	x	8.0 Gpa	/	1.17 m	)=	123.5E+3 Nm2/m
Battens			268.0E+3	x	8.0 Gpa	/	0.40 m	)=	5.4E+3 Nm2/m
Ceiling	0.45 m	x	857.4E+0	x	3.0 Gpa	/	12	)=	96.5E+0 Nm2/m

Total Transverse Flexural Stiffness Dy = 131.6E+3 Nm2/m

Stiffness Ratio Dy/Dx = 0.163

Dynamic Properties - in X (ie Joist) Direction

Joist response

Midspan Joist Deflection
$$\Delta j = \frac{5w_jL_j^4}{384E_jI_j}$$
= 1.7 mm = Span / 2000

Fundamental Frequency
$$f_0 = \frac{C}{\sqrt{\Delta_s}} = \frac{17.7}{\sqrt{\Delta_s}}$$
= **13.38 Hz** > **8.00 Hz**

Performance Evaluation

This Response Criteria is appropriate - Continue

$$N_{40} = \left(\frac{B}{L}\right)\sqrt{\sqrt{\frac{\left(\frac{D_d}{D_j}\right)+\left(\frac{40}{f_0}\right)^2-1}{\left(\frac{D_d}{D_j}\right)}}-1}$$
= 5.62

Max Unit Impulse Velocity, Vmax
$$u_{velm\max} = \frac{4000(0.4+0.6N_{40})}{\left(\frac{wBL}{g}+200\right)}$$
= **6.66 mm/N-sec2**

For normal residential occupancies ξ = 0.02

Then $\sigma = \xi f_0$ = 0.02 x 13.38 Hz = 0.2676

Good Performance

Require Vmax<2*100$^{\sigma o}$ < **6.86 mm/N-sec2**

Performance Doubtful

Require Vmax<3*100$^{\sigma o}$ < **10.29 mm/N-sec2**

Clearly Unacceptable Performance

Require Vmax<4*100$^{\sigma o}$ < **13.72 mm/N-sec2** Deflection = Span / 2000

Calculated V max = 6.66 mm/N-sec2 Frequency = 13.38 Hz

Conclusion Good Performance

7.6 Horizontal Vibrations

Horizontal vibrations that cause problems usually involve movement of the entire structure, rather than the component parts as would be normal for vertical vibrations. The problems encountered are usually ones of annoyance and human discomfort rather than structural distress and are often created by wind or machinery generating an excitation frequency similar to the natural frequency of the building. Such movements are usually tolerable provided the frequency of occurrence is small (i.e. when generated by rare events such as very strong winds or earthquakes). When common events such as moderate wind storms result in substantial lateral horizontal movement, the serviceability of the building is generally affected and a problem is recognised.

Human tolerance levels to axial (foot-to-head) vibration has been found to be lower than the tolerance to front to back vibration at an excitation frequency of greater than 3 Hz. Thus most complaints result from vibration when the person is in a reclining position (i.e. in bed or lying down). This is as a result of the full body contact achieved in this position. Consequently the design criteria applied to vertical vibrations can be partially applied to horizontal vibrations in this situation.

The majority of complaints received are applicable to houses (3 to 12 Hz), rather than taller buildings (0.2 to 1 Hz), which are remote from the period of fundamental frequency for humans (5 Hz). The most common form of construction with a fundamental frequency in this range is the pole house (3.8 to 7-Hz). Such structures are often constructed in very exposed locations and are thus subjected to stronger wind loads more frequently. The long poles used on steep sites are usually flexible and as a result have a low fundamental frequency. The installation of a sufficient number of well-fixed cross-braces provides a means of overcoming the vibration complaints. It usually requires cross-bracing which is in excess of that required for strength considerations in order to provide the necessary rigidity against horizontal movement.

Some modern high rise buildings, particularly those which are relatively slender, can experience annoying lateral movement, usually induced by wind effects. Restaurants which are often located at the top of towers are particularly prone to experience such problems. In these cases, people are sitting, and therefore are in good body contact with the building. Cutlery and crockery are very good secondary indicators of side-sway. Cenek & Wood (1990) suggested the side sway acceleration limit of 1% g (item S16 of Table 1) based on work by Galambos (1973) and Melbourne and Cheung (1988).

8. CONCLUSIONS

The serviceability limit state criteria contained within this report are at best fuzzy limits which 'should prevent problems in most usual applications'. Designers using the values contained herein should use their discretion and judgement to adjust them according to the specific circumstances of the application under consideration. In truth, while some criteria can be measured (e.g. crack widths) these are usually residual effects from a previous loading history. Similarly impact loads generated by the knocks and bumps expected during normal building use are transient and their realistic quantification difficult to replicate.

As mentioned in the text, criteria to avoid floor liveliness and vibration problems are the subject of ongoing study, with field measurements of the dynamic characteristics of known problem floors currently under way (1999) and an additional study planned. The criteria outlined in this report are in use overseas and are apparently giving designers some guidance by which floor liveliness is being controlled. Work continues in this area.

9. REFERENCES

Allen, D.E., Rainer, J.H., Pernica, G. 1985. Vibration Criteria for Assembly Occupancies. Division of Building Research, Paper No 1310, National Research Council of Canada, Ottawa, Ontario.

Allen, D.E., 1990. Building Vibration from Human Activities. Concrete International: Design & Construction, Vol 12 (6) pp 66-73, American Concrete Institute, Washington.

Allen, D.E., 1990a. Floor Vibrations from Aerobics. Canadian Journal of Civil Engineering, Vol 17 No.5 p771-779, Ottawa, Ontario

Allen, D.E. & Murray, T.M., 1993. Design Criteria for Vibration Due to Walking, Engineering Journal of the American Institute of Steel Construction, AISI, Fourth Quarter, pp. 117-129.

Allen, D.E. and Rainer, J.H. 1976. Vibration Criteria for Long-span Floors. Canadian Journal of Civil Engineering. Vol. 3, No.2 pp 165-173.

American National Standards Institute (ANSI), 1988. Building Code Requirements for Minimum Design Loads for Buildings and other Structures, ANSI/ASCE 7-88, American Society of Civil Engineers, New York.

American Concrete Institute (ACI), 1974. Deflections of Concrete Structures. ACI Publication SP43. Detroit, Michigan.

Bachmann, H. & Ammann, W. 1987. Vibrations in Structures: Induced by Man and Machines. IABSE-APIC-IVBH. Zurich.

Beattie, G.J. 1998. The vibration performance of timber floors. BRANZ, SR 79. Judgeford.

Berrill et al, 1980. “Study Group Report on the Seismic Design of Bridges”, Bulletin NZ National Society of Earthquake Engineering, Vol 13(3), Sept. 1980

Building Research Association of New Zealand (BRANZ), 1991. Designing for Thermal & Moisture Movement, Bulletin No 285, Judgeford.

Carson J., 1994. Deflection Criterion to ensure an acceptable level of dynamic performance from floor joists systems. Unpublished report by National Forest Industries Association, Canberra, Australia.

Cenek P.D. & Wood J.H., 1990. Designing Multi-storey Buildings For Wind Effects. BRANZ Study Report No. 25, Judgeford.

Constructional Steel Research and Development Organisation (CONSTRADO) 1983. “Steel Designers Manual”, London.

Eurocode (EC) 5, 1993. Design of Timber Structures. Part 1.1 General Rules and Rules for Buildings. European Commission on Standardisation. Brussels.

Galambos, T.V. (1973). Structural Deflections - a literature and state-of-the-art survey. National Bureau of Standards. Building Science Series 47, Washington DC, October

International Standards Organisation (ISO), 1989. Evaluation of Human Body Exposure to Whole-body Vibration - Part 2: Human Exposure to Continuous and Shock-Induced Vibration in Buildings (1 to 80 Hz), ISO 2631-2, Geneva.

International Standards Organisation (ISO), 1992, Basis for the Design of Structures - Serviceability of Buildings Against Vibration", ISO 10137, Geneva.

Ellingwood B., et al 1980. Development of a Probability Based Loading Criterion for American National Standard A58. NBS Special Publication 577, June 1980, Washington.

Melbourne. W.H. and Cheung, J.C.K, (1988). Design for Serviceability Accelerations in Tall Buildings. 4th Int. Conf. on Tall Buildings, Hong Kong & Shanghai, April/May.

NBC, 1990. Supplement to the National Building Code of Canada 1990. Commentary A on Part 4 of the National Building Code of Canada 1990, Serviceability criteria for deflection and vibration. National Research Council of Canada, Ottawa, Ontario.

New Zealand Government, 1992. Building Regulations 1992. New Zealand Government Printer, Wellington.

Ohlsson S., 1988. Springiness and human induced floor vibrations - A design guide. Swedish Council for Building Research, Stockholm, Sweden.

Rainger P., 1984. Movement Control in the Fabric of Buildings. Batsford Academic & Educational Ltd., London.

Roake R.J. 1975. Formulae for Stresses and Strains. McGraw-Hill, 5th Edition, New York.

Standards Australia (SAA), 1989. Minimum design loads on structures. AS 1170, Sydney.

Standards Australia (SAA), 1992. Domestic metal framing. AS 3623, Sydney.

Standards New Zealand (SNZ), 1990. Code of Practice for the Design and Construction of Masonry, NZS 4223, Wellington.

Standards New Zealand (SNZ), 1992. Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203, Wellington.

Standards New Zealand (SNZ), 1993. Structural Timber Design, NZS 3603, Wellington.

Standards New Zealand (SNZ), 1995. Structural Concrete Design, NZS 3101, Wellington.

Steffens, R.J. 1974. Structural Vibration and Damage. Building Research Establishment, Watford, UK.

Thurston, S.J. 1993. Report on racking resistance of long sheathed timber framed walls with openings. BRANZ, SR 54. Judgeford.