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SERVICEABILITY CRITERIA FOR BUILDINGS

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BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

PREFACE

Building structures must be designed for strength and serviceability. In general, building control documents concentrate primarily on strength requirements. The use of higher strength materials has led to lighter structures, larger deformations and correspondingly some serviceability problems.

The authors believe there is a lack of adequate guidance on allowable deflections for structural designers in New Zealand. This document is intended to assist structural engineers establish suitable deflection criteria, in order to ensure serviceability of buildings. It consists of deflection limitation tables, and a commentary.

ACKNOWLEDGEMENTS

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The Association also acknowledges the contribution from Allen, Rainer and Pernica in their paper 'Vibration Criteria for Assembly Occupancies' (Bibliography 2-13). This paper formed the basis of section 3 'Dynamic Effects' with the permission of the authors.

This report is intended for structural engineers and designers.

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KEYWORDS

From Construction Industry Thesaurus - BRANZ edition: Buildings; Creep; Damping; Deflection; Deformation; Durability; Dynamic loads; New Zealand; Structural design; Vibration: Vibrational stress

ABSTRACT

The increasing use of high strength materials in building constructions often results in relatively flexible components which are more prone to deflection problems. Such problems manifest themselves in various ways, including annoying vibrations, unsightly misalignment and, albeit infrequently, structural failure.

Deflection is recognised as a serviceability limit state. Several limits are identified relating to aesthetics, felt movement, vibration, cracking of finishes, and potential structural damage. The qualitative assessment of acceptable deflection limits is presented in this report. Methods of analysis for deflection and limits applied overseas are considered.

Deflection limits and the controlling criteria are presented in the form of tables which correlate to the location and function of components. The report discusses the parameters assumed when determining deflection (degree of end fixity, E and I values, loading regime, combined displacements) and the sensitivity of the calculated deflection to these variables.

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INTRODUCTION

A building must not only have sufficient strength to carry the maximum expected loads with an adequate factor of safety, but deflections and movements under working loads must not be so large as to cause damage or result in the building being unserviceable in any way.

Prior to the 1960s allowable design stresses were low and design methods were conservative. The resulting buildings tended to be massive with comparatively short spans. They were stiff enough to preclude most deflection problems, and building codes of that era quite justifiably dealt mainly with working stresses.

More recently, an increased understanding of structural behaviour and material properties has permitted the use of strength methods of design, increased allowable stresses and provided for the greater use of higher strength materials. Also, constructions including prestressed and precast components and lighter non-structural components have removed many structural redundancies. The resulting buildings, with more slender members and longer spans, are correspondingly more flexible and therefore prone to larger deflections and consequent unserviceability.

Building serviceability may become seriously affected with cracking, movement and vibration problems consequential to the more flexible structure. Nevertheless, building control documents tend to place emphasis on adequate strength control.

This document is intended to assist structural engineers establish suitable deflection criteria in order to ensure serviceability in buildings.

USE OF THIS DOCUMENT

Tables of deflection limitations for three types of building components are provided. These are:

1. Horizontal components (beams, floors, roofs, rafters)
2. Vertical components (walls, columns)
3. Domestic components

Each table gives the reasons for which deflection should be limited, the suggested deflection limit, and the corresponding load combination. Where appropriate, examples and comments are given.

The commentary gives further background information and examples.

When using the Tables, the following factors need consideration:

1. Serviceability criteria are subjective and do not strictly define limits. The recommended deflection limits given in the tables are those that are likely to be acceptable to the majority of people using the building.

2. The limits given in Tables 1 to 3 are applicable to normal domestic and commercial buildings. The criteria may be relaxed where people's expectations for serviceability are less (e.g. in farm buildings, sheds, horticultural buildings - particularly where these are unlined and the potential for damage to non-structural elements is minimal). The Canadian National Building Code (Bibliography 1.11), recommends that the visual deflection limits as outlined in the Tables be increased by 50% for such buildings. Increases of at least this amount seem appropriate for use in New Zealand.

3. The building must satisfy the appropriate control document requirements with respect to strength, and where stated, with respect to deflection. Where the control documents refer to 'acceptable serviceability requirements', the deflection limitations as outlined in the Tables are intended to provide a means of compliance with such requirements.

4. The designer should decide on the reasons that the deflection is to be controlled for each element. There may be several reasons, each of which may have a different deflection limitation. The loading combination which coincides with the particular deflection control criteria is required to be applied to the most critical combination for both the element and any secondary elements which may influence the deflection. The deflection limits outlined under the 'sensory acceptability' are independent from deflections generated elsewhere and only the load case specified need be considered. In other cases, the long-term creep deflections from dead local plus sustained live load will require to be added algebraically to the short term deflections from live or other temporary loads. The combined deflection is to be less than the deflection limitations outlined.

5. Fixing details must be consistent with both the stress distribution assumed during analysis and the deflection mode assumed. Just as secondary stresses resulting from end fixity conditions are required to be considered when assessing required component strengths, so relaxation resulting from end fixity is required to be considered when assessing deflection.

6. When assessing the deflection of components, the sensitivity of the assumptions made with respect to the following aspects should be considered:

   a) Section modulus
   b) Changes in section (notches, composite sections, etc.)
   c) Component end restraint and rotation effects
   d) Flexibility of supports (combined deformation, rotation effects)
   e) Loading assumptions (intensity and distribution of loads)
   f) Duration of load (creep effect)
   g) Environmental effects (thermal expansion/contraction, moisture movements & shrinkage)
   h) Shear distortions

TABLE 1: HORIZONTAL COMPONENTS (BEAMS, FLOORS, ROOFS, RAFTERS)

REASONS FOR LIMITING DEFLECTIONS		DEFLECTION LIMITATIONS	LOAD COMBINATION CAUSING DEFLECTION	EXAMPLES AND COMMENTS
1. Effect on Structural Elements	(1) Instability or change in primary structural system	check that deflections do not - (i) cause instability (ii) alter primary structural system	combination of D, L, W, Eq	Note: This table covers roofs and rafters at any angle to the horizontal, as well as beams and floors
	(2) Water accumulation (ponding) on roofs etc	d < 1/250; for beams parallel to line of roof slope	D (allow for creep) plus rainwater or snow melt	
	(3) Second order effects			
	(a) differential thermal expansion	d < 1/300; < 15 mm check side wall capacity		differential expansion of exterior beams of air-conditioned building may cause bending. restrained movement results in membrane secondary stresses. Affected by colour, length, solar exposure and inclination.
	(b) differential settlement	d < 1/300 d < 1/150		- Beams supporting masonry walls - Beams supporting walls other than masonry - Note it is the differential, not total settlement which is usually important.
2. Effect on Sensory Acceptability	(1) Visual vertical sag			When calculating deflections consider - effective section moment inertia - effective end restraint - environment moisture/temp - load duration (creep) sag = net deflection under permanent long-term load = d - camber Ls = sustained live load. The designer must determine the proportion of the live load which is sustained for long periods.
	(i) along soffits and invert of beams on line of sight	sag<1/500	D + Ls	
	(ii) across soffits and inverts of beam visible from the side	sag<1/250	D + Ls	
	(iii) busy floors, roofs & rafters	sag<1/180	D + Ls	
	(iv) specialist floors	sag<1/360	D + Ls	specialist floors using trolleys etc.
	(2) Perceptible movement (a) vibration due to vertical dynamic loads (i) resonance of beams supporting floors subjected to vertical dynamic loads			Refer to page 22
	(ii) annoyance factor limitations	d(stat)< 1.0 mm for floors	Lc = 1 kN	This limitation is intended to insulate people from sensing others moving on floors. The limitation should also enable a person to walk on a floor without noticing undue movement.
	(iii) liveliness control	d(stat)< 1/360	L = full live load at any point	This is a lower bound limit to prevent a floor from being too springy.
	(b) deflection due to wind	d< 1/300	W	Movement due to wind uplift on purlins and rafters which can be seen. When rafters and purlins are hidden from view or well removed, no deflection criteria are necessary. Note: whilst this criterion relates to a 50 year average return period wind, it is such that the deflections will be acceptable during more frequent storms having average return periods of 10-12 years, acknowledging the maximum event will occur more often.

REASONS FOR LIMITING DEFLECTIONS		DEFLECTION LIMITATIONS	LOAD COMBINATION CAUSING DEFLECTION	EXAMPLES AND COMMENT
3. Effect on Use	(1) Beams that support surfaces which should drain water	d<1/250 d<1/350 d<1/600	D + L or D + S D + L or D + S D or D + S	- reinforced concrete or steel beams supporting slabs - trafficable deck supported by timber beams - non-trafficable deck supported by timber beams always check that water flows as designed
	(2) Beams should remain plane:			
	(a)direct use of the surface	d(add)*< 1/600	D + Ls	- e.g., gymnasiums, squash courts, bowling alleys.
	(b)effect of beam deflection on doors and opening windows	d(add) < 1/240 and < 25 mm	D + Ls	- e.g., jamming of doors and windows depends upon clearances
	(3) Members support sensitive equipment	d(add) < manufacturers recommendations	D + Ls	
4. Prevention of Damage to Non-Structural Elements	(1) Supported walls			
	(a)masonry and plaster	d(add) < 1/500 and < 0 mm d< 1/300	D + Ls	for deflection calculations assume equipment is part of dead load. examine loads which cause deflections after equipment is installed and levelled, i.e., creep caused by dead and sustained live load and immediate deflection under moving live load
	(b)Moveable partitions	d(add)< 1/250 and < 20 mm	D + Ls	
	(2) Ceilings			
	(a)plaster or Gibraltar board	d(add) < 1/360 and < aa20 mm	D + Ls	
	(3) Surfaces:			
	(a)floor finishes	d< 1/180	D + L	ensure that calculated differential settlement does not exceed this limitation.
	(b)roof coverings	d< 1/180 < 1/60	D + L W	
	(c)facades/curtain walling	d< 1/180	D + L	e.g., uplift of ceramic tiles
	(d)Fixed glazing	d < 2 x clearance of glass in frame (approx 10 mm)		no limit necessary if sufficiently isolated
	(4) Damage to bearing zone of structural masonry walls and columns	d< 1/300	D + L	A simply supported beam can rotate the top course of a masonry wall or column
	(5) Vibrations causing damage	d(dyn) < 3 mm where: $d(dyn) = d(stat) x K_1 K_2$	Dynamic load L(dyn)	$d(dyn) = d$ (stat x K_1 x K_2 $K_1 = L(dyn)/(D+L)$ $\quad K_2 = \frac{1}{1 - (f/fo)^2}$
	(6) Thermal movements	**	60°C variation	e.g., in swimming pools thermal contraction of supporting beams due to filling the pool with cold water can damage tiles.
	(7) Moisture Movement	**		
	(8) Frames or trusses adjacent to rigid wall (vertical or horizontal movement)	d< $\frac{1}{250}$ x frame spacing to wall	W	

* (add) = additional deflection occurring after installation of partitions

** Thermal and moisture movement limits are dependent on the compatibility of linings and control joint capability

TABLE 2: VERTICAL COMPONENTS (WALLS AND COLUMNS)

REASONS FOR LIMITING DEFLECTION	DEFLECTION	DEFLECTION LIMITATION	LOAD COMBINATION CAUSING DEFLECTION	EXAMPLES AND COMMENTS
				NOTE: Where specific references are not given, refer to the corresponding sub-section in TABLE 1, BEAMS etc
1. Effect on Structural Elements	(1) Instability or change in primary structural system	Check that deflection does not:		
		(a) cause instability	Full working load	in columns, lateral deflections can lead to P-delta effect
		(b) alter primary structural system	Full working load	walls are usually too stiff in-plane for this to be a problem
	(2) Second order effects			
	(a) differential thermal expansion			e.g., if roof expands, side walls can be forced outwards, causing cracking
	(b) differential settlement	$d < h/300$ (masonry) $d < h/150$ (other materials)		ensure differential settlement does not exceed this limitation thermal shortening of external columns is treated similarly
2. Sensory Acceptability	(1) Visual			whilst visible leaning of walls is unacceptable, it is usually a problem of contruction tolerances rather than deflection
	(2) Perceptible movement (a) lateral deflection of walls due to wind in-plane loads	$d < h/300$	D + W	although load factors have not been specified in the 'load combination' column, loads should correspond to those specified for working stress design in the appropriate building control documents. (e.g., 0.7D + W) - Note: Movement of walls during earthquakes is not considered important. - dynamic case
	face loads	$d < h/180$	D + W	- walls in lowrise buildings
		$d < h/500$	D + W	- glazed panels in high rise structures
	(b) sway of columns due to wind	$d < h/500$ and < 4 mm per storey	D + W	applies especially to multi-storey buildings, often requiring special study. Note that perceptible sway during earthquakes is considered acceptable
	(c) frame deflection due to wind and earthquake	Horizontal deflection at eaves $d < 1/200 \times$ frame spacing and < 40 mm in end bay	W	applies to portal frames. Because of the flexible nature of portal frames under lateral loads, all brittle non-structural elements should be adequately separated from the portal
3. Effect on Use	Sway of structures supporting sensitive equipment.	Comply with equipment manufacturers specification	loads applied after equipment is installed	e.g., microwave and other transmitters/receivers mounted on roofs

REASONS FOR LIMITING DEFLECTION		DEFLECTION LIMITATION	LOAD CONTINUATION CAUSING DEFLECTION	EXAMPLES AND COMMENTS
4. Prevention of Damage to Non-structural Elements	(1) Walls			
	(a) in-plane loading			
	masonry and plaster	$d < h/500$ and < 10 mm		check NZS 4203 requirements also
	moveable partitions	$d < h/500$ and < 25 mm		
	(b) face loading			
	masonry and plaster	$d < h/500$	D + W	Note: whilst this criterion relates to a 50 year average return period wind, it is such that the deflections will be acceptable during more frequent storms having average return periods of 10-12 years, acknowledging the maximum event will occur more often.
	moveable partitions	$d < h/200$	D + Eq	
	(2) Surfaces			
	(a) in-plane loadings facades/curtain walls	$d < h/150$		in certain instances compliance with NZS 4203 will dictate.
	fixed glazing	$d < 2 \times b$ clearance in frame and < 10 mm		
	(b) face loading Gib. and plaster	$d < 1/500$	W or Eq	
	Other linings	$d < 1/250$	W or Eq	
	(3) Vibrations	$d(dyn) < 3$ mm	Dynamic Load (L dyn)	e.g., machinery fixed to walls or columns $d(dyn) = d(stat) \times K_1 \times K_2$; $K_1 = L(dyn) / (D + L)$; $K_2 = \frac{1}{1 - (f/fo)^2}$
	(4) Thermal Movement			

TABLE 3: COMPONENTS OF DOMESTIC BUILDINGS

COMPONENT		DEFLECTION LIMITATION	LOAD COMBINATION CAUSING DEFLECTION	EXAMPLES AND COMMENTS
1.Bearers		d < 1/360	D + L	dead loads and floor live loads only. Upper story loads assumed to be carried by bearer supports via wall deep beam action.
2.Joists		d < 1/360 and < 9 mm	L	liveliness control
		d < 1/300 and < 12 mm	D + Ls	- i.e., sustained live loads.
3.Lintels	(a) Supporting roofs and ceilings	d < 1/250	D	- limit to 12 mm if soffit on line of sight.
	(b) Supporting floors	d < 1/300 and < 9 mm	D or L	- limit of 9 mm is to insulate people from sensing others moving on floor, (i.e., liveliness control)
4.Rafters	(a) Light roofs	$d<1/500(1^2/(12.4x10^6))$ (where d,1, in mm)	D	e.g., roll formed steel or aluminium, pressed metal tiles, membrane roofing on ply
	(b) Heavy roofs	$d<1/250-(1^2/(5.8x10^6))$ (where d, 1, in mm)	D	e.g., concrete or clay tiles.
5.Ceiling Joists, Ceiling Runners Strutting Beams		d < 1/360	D	limit to 12 mm if soffit on line of sight
6.Structural Beams e.g., Beams supporting roofs, ceilings, and/or floors	(a) Floor members	d < 1/300	D	
		d < 1/360 and < 9 mm	L	9mm limit is to insulate people from sensing others moving on floors i.e., liveliness control. 9 mm controls on longer spans (1>3.2 m) when more than one person is likely to be on each span.
	(b) Roof members	d < 1/300 and < 20 mm	D	
		d < 1/300 and < 12 mm	L	
7.Walls	(a) In-plane loads	d < h/300	D + W D + E	
	(b) Face loads	d < h/180	D + W	

Note: Table 3 is based on traditional New Zealand light timber frame home building practice and the deflection limitations are derived from unpublished notes held by the Building Research Association of New Zealand, Judgeford, N.Z. For special designs, controls of vibrations, resonance etc, refer to the relevant Table.

Note: Use load factors for working stress design from appropriate building control documents (e.g,. 0.7 D + W, 0.7D + 8E)

COMMENTARY

1. Reasons for Limiting Deflections

Excessive deflection of components can be manifest in damage to structural elements, unacceptable sensory effects, detrimental effects on use, and damage to non-structural elements. The presence, of some, any, or all of these effects reduces serviceability accordingly.

1.1. Effect on Structural Elements

Excessive deflection leading to damage to major structural elements could render the building unsafe. Deflections of any component should therefore not be great enough to alter the primary structural system or cause instability of the structure. The effects of deflections on the distribution of forces and on loadbearing capacity must be considered in structural calculations.

The following examples illustrate this point:

Example 1: Creep and Shrinkage

In an actual case, shrinkage of a floor slab together with creep deflection at midspan, caused rotation of the torsionally weak edge beams. The structural steel columns were integral with the beams and were subjected to inwards movement and rotation sufficient to bend the columns to such an extent that collapse was feared (see Figure 1).

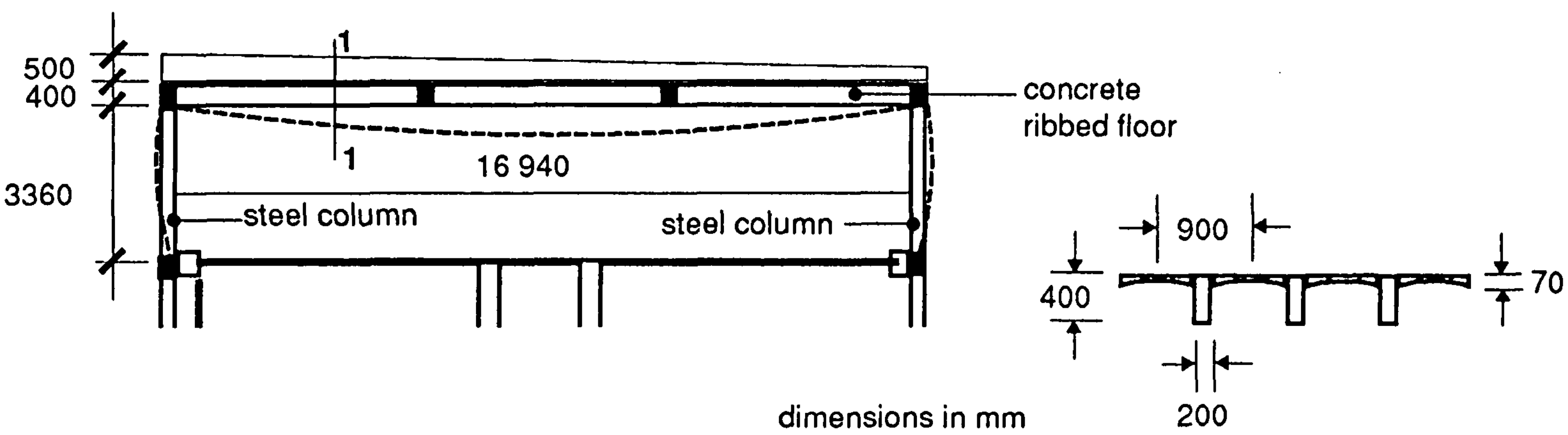


Figure 1 : Shrinkage and creep deflection in a floor system.

For methods of calculating shrinkage and creep, refer to Commentary, Section 2.5.2 and 2.6.

Example 2: Slab deflections

1.5m deep precast cladding panels were supported at their top edge by four cleats fastened to a 175 mm thick floor slab edge beam. The beam deflected under imposed live load causing the two inner support points to carry little load, thus overstressing the two outer supports.

Example 3: Ponding

The problem of water ponding on flat roofs which have a small slope has caused many serious failures. The structure deflects under its own weight which can cause small ponds to form. The perimeter drainage points then become relatively high points of the roof. Normally the depth of ponding will increase until discharge occurs at the drainage points provided and equilibrium is restored. If the roof is too flexible it will deflect further under the increasing weight of the accumulated water until it collapses. See Figure 2.

Ponding is prevented when the camber of a flat roof is sufficient to counter its selfweight deflection (i.e. 1.3% slope is sufficient for a deflection limit of span/250). This requirement applies to the combined deflection of both primary elements (beams) and secondary elements (rafters) elements. See Recommended Deflection Considerations, page 12.

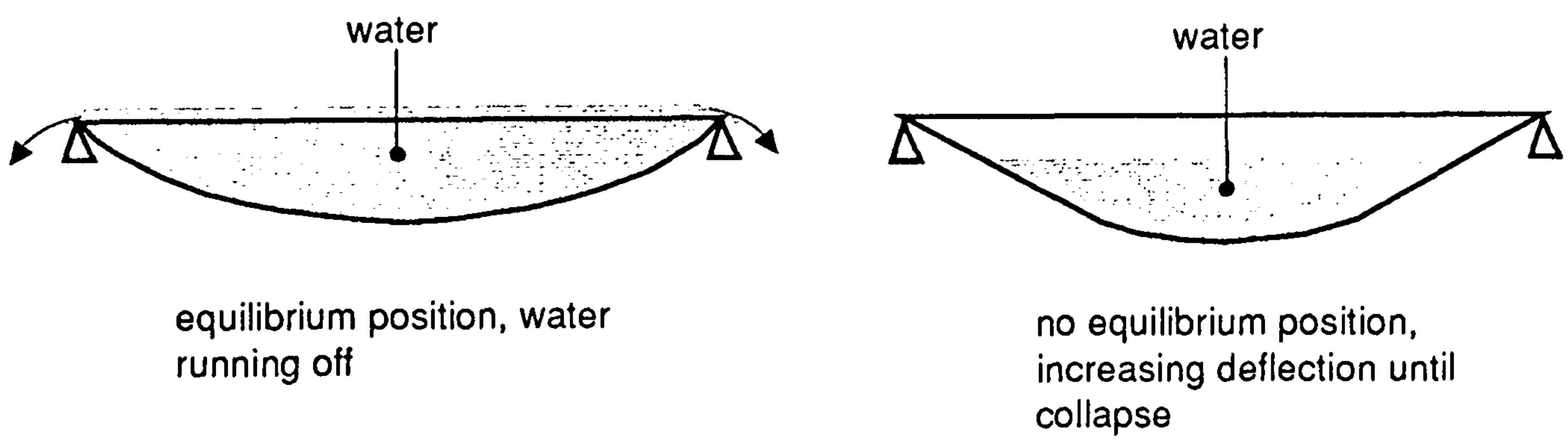


Figure 2 : Deflection through ponding on a flat roof.

1.2 Effect on Sensory Acceptability

The results of several studies show that excessive deflections of spans and connections do not look attractive to the public and are often thought to be unsafe. These observations are subjective, and although the deflections will not endanger the structure nor shorten the structure's lifetime, it is appropriate to limit deflection to ensure confidence is maintained. Such limits apply to the long-term deflection. The use of a camber where possible and practicable should help to reduce the sag due to long-term loads. Refer to Appendix B (page 31) for suggested cambers for the various components and building materials.

Dynamic loads may cause perceptible or annoying vibrations. These vibrations make the user feel uneasy. Examples are the motion of slender multi-storey buildings during strong wind gusts, or people feeling the vibrations caused by other people walking on floors.

If periodic forces are synchronised with a natural frequency of the building or component, the vibrations can be greatly amplified. This condition is called resonance and should be avoided.

The undesirable effects of continuous vibrations caused by machines can be minimised by special design provisions such as vibrational isolation of the machinery, physically relocating the machinery away from deflection sensitive locations or occupancies within the building, or alteration of the frequency of the structure. The latter is difficult to achieve and the least preferred option.

For more detail, see **Dynamic Effects**, page 19.

1.3 Effect on Use

Excessive deflections in a structure may have a detrimental effect upon its use. This can vary from annoyance to the users, to the inability of the structure to perform as intended. Remedial work can be expensive.

Excessive deflections, faulty detailing, or insufficient fall on roof terraces, balconies and roofs may render these areas virtually unusable for long periods of time because of the formation of puddles after a rainfall.

In certain instances, deflections may have a detrimental effect on the opening and closing of large doors and windows. It may also be difficult to shift purpose-made moveable partitions.

Other surfaces, such as bowling alleys and gymnasiums, must remain plane for practical reasons, whilst those surfaces supporting sensitive equipment should be designed to the specifications provided by the equipment manufacturer or user.

1.4 Prevention of Damage to Non-Structural Elements

Deflection limitations should be applied to buildings and their components to avoid non-structural elements being introduced into the load path. If there is insufficient separation between deflecting primary structural elements and non-structural components, the load may be transferred through the non-structural element. Such non-structural elements may include walls, partitions, windows, ceilings, floor and roof coverings, facades, service pipework, lifts, stairs, etc.

The resulting damage may include cracking and buckling of some elements of moderate stiffness and integrity, or tearing and folding of more flexible items such as coverings and linings. Whereas these forms of failure may not effect the stability of the structure, they may render the building unserviceable by damaging the functional aspects of the structure, (i.e., aesthetics, weathertightness, thermal and sound insulation).

Non-structural elements are normally installed after the self-weight deflection of the structure has occurred. Thus the immediate self-weight portion of the deflection need not usually be considered when determining separation requirement. However, the additional deflection as result of permanent imposed load, total creep, and temporary live load may cause damage to elements that have been installed prior to such deflection occurring. It is this secondary deflection that causes damage to non-structural components.

Recommended Deflection Considerations

2. General

A comprehensive list of simple deflection formulae is presented in Reference 4.6. Designers should be aware of compounding deflections where deflections of primary elements may superimpose both linear deflections and magnified rotational deflection upon supported secondary elements. (Refer example, section 2.4).

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

(a) Modulus of elasticity (E) - refer section 2.1
(b) The effective moment of inertia (I) - refer section 2.2
(c) The effective end restraints - refer section 2.3
(d) Settlement and rotation of the supports - refer section 2.4
(e) Environmental effects (i.e., temperature and moisture) - refer section 2.5

(f) Time and duration of load application (i.e., creep) - refer section 2.6

Table 6 (page 15) has been prepared to identify the deformation characteristics for most materials used in building components. The table includes typical values of the modulus of elasticity, thermal expansion coefficients and water absorption characteristics of these materials.

2.1 Modulus of Elasticity (E)

The modulus of elasticity is the ratio of the stress to strain of a material while it deforms elastically. Whereas some materials have a constant value of E (e.g., steel), other materials (e.g., cement products, plastics, etc.) may have a value of E which will depend upon their composition, strength, temperature, moisture content, or other parameters. Where appropriate the range of the E values for this latter type of material are given in Table 6 and the designer is required to gauge the significance of the selected value of E used to compute a particular deflection. Long-term static load deflection calculations sometimes accomodate the effect of creep by modifying the value of E. (See section 2.6).

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

The moment of inertia is a geometric property of the section. Some materials (e.g., steel, timber, etc) behave in a similar manner under tension or compression. Other types of material may be quite different, and may rely on composite action to resist load (e.g., reinforced concrete, glass reinforced cement). In such cases, cracking of the section often occurs at low tensile stresses, with other materials being used as composites to resist these applied forces. Where cracking occurs, the effective bending stiffness is that of the transformed section derived by combining the moments of inertia of the components and their respective modulus of elasticity for each component. The designer should be aware, however, that the extent of cracking is dependent on the applied moment and the axial load, both of which may vary along the length of the member.

For reinforced concrete beams it is suggested that, for analysis, the moment of inertia be 50% of that of the uncracked section. For reinforced concrete columns, subjected to axial load and moment, the effective moment of inertia of the uncracked section is in the range 100% (for columns subjected to high axial compression) to as low as 40% or less (for columns subjected to low axial compression or tension).

For an accurate method of calculating deflection of reinforced concrete beams using beam curvatures, and worked examples see MacGinley (Bibliography 3.5).

2.3 Effective End Restraint

Bending formulae are usually based on the assumption that plane sections remain plane during bending. While this is correct for members of normal proportions, it is not appropriate when considering the end joint zones of fixed-ended members. The stiffness of these joint zones may be significant and should be taken into account when applying the deflection formula or during the analysis.

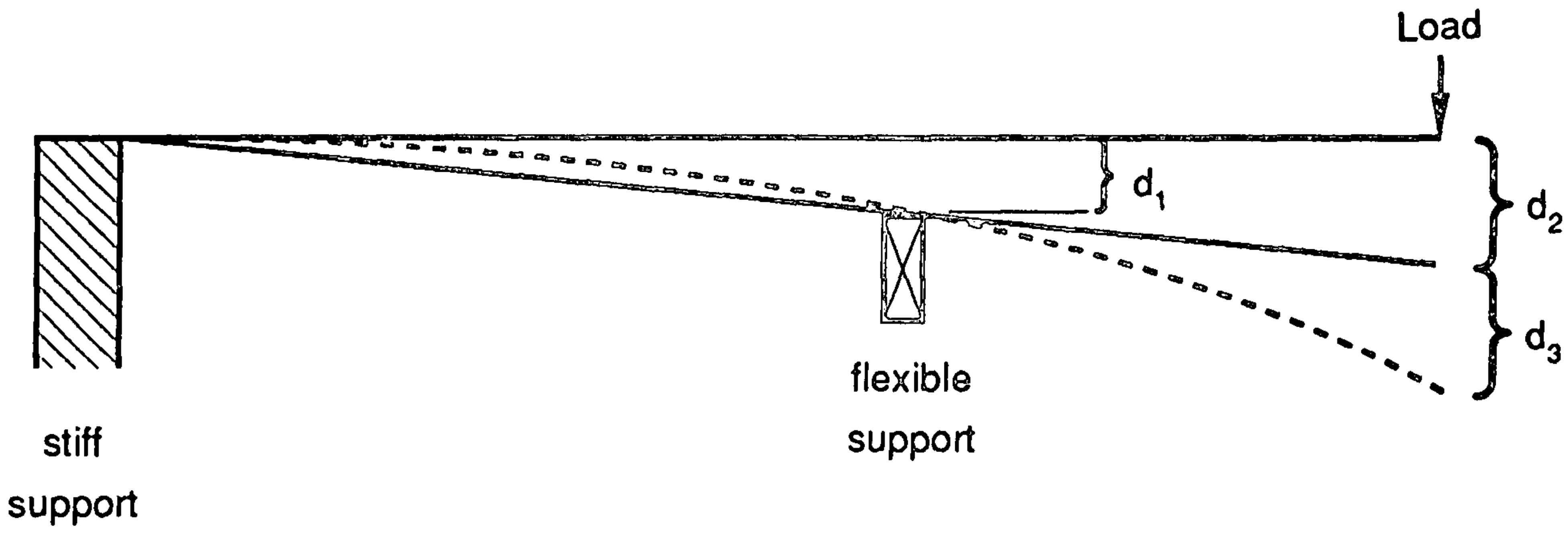
2.4 Flexibility of Supports

The simple deflection formulae assume that the supports are rigid. Where elements are supported by other elastic components, or on foundations which are not rigid, two dimensional analyses may model the supports as springs with the support deflection dependent on the loads imposed.

Example: A grid of floor joists is supported by a wall at one end and another support, a beam, beyond which they cantilever. To arrive at the total deflection of the free end of the joists, see Figure 3, page 13.

2.5 Environmental Influence on Deflection

All materials experience dimensional change when subjected to changes in



d_1 = Relative displacement between the two supports.
d_2 = Displacement of free end resulting from d_1.
d_3 = Displacement of free end resulting from cantilever flexure.
Note: All deflections may have elastic and plastic components.

Figure 3 : Deflection of a cantilevered floor joist grid on elastic supports .

temperature and moisture content. The degree of dimensional change varies markedly with different materials. The changes are often cyclic in nature; however, the original dimensions may not be restored when the environment returns to its initial condition.

Within the normal service conditions of buildings, thermal movements will exceed moisture related movement for most materials. However, for cement-based products the extent of movements is similar for both effects, while for brick and wood-based products moisture movement is likely to be more significant than thermal effects.

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 removes moisture resulting in shrinkage of the component), it is acceptable practice to design for only the most severe movement.

Potential movement may be accommodated by any of the following:

1. provision of adequate control joints to accept the movement
2. suppression of the movement by translating it to internal stresses and reactions of the restraining mechanism
3. minimising the movement by employing design techniques such as changing materials, application of coatings to reduce moisture absorption, or selecting appropriate colours to reduce solar induced thermal effects.

2.5.1. Thermal Movements

Most materials expand when heated and contract when cooled. 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.

Table 4 shows how different colours affect the maximum temperature that galvanised steel roofs have been found to attain. An alternative approach, as shown in Table 5, gives some indication of how different materials build up and lose heat over the ambient air temperature through

Table 4: Colour-Temperature interaction for galvanised steel roofs

Surface Colour	Maximum Roof Surface Temperature °C
black	70
red	63
unpainted	60
aluminium (paint)	50
cream	48
white	44
	Ambient Air temperature=20°C

solar gain or night loss. The inclination and exposure of the components is not incorporated in this approach. When used in conjunction with Meteorological Service temperature records, this alternative approach provides a method of assessing temperature extremes where unusual conditions may be envisaged (e.g., Central Otago, at high altitude). The coefficients of linear thermal expansion for many materials are listed in Table 6. 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. Conversely, for light-coloured, well-ventilated panels the range may be less. Table 7 indicates the range of temperatures measured for various combinations of material colour, material and insulation.

Tables 4 and 5 contain temperature information derived from measurements throughout New Zealand. Further background on these tables may be found in BRANZ Bulletin 238 "Sealed Joints in External Claddings - 1. Joint Design."

Table 5: Temperature variations of materials

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: Deformation properties of building materials

Material Type	Linear Thermal Movements			
	Modulus of Elasticity, x 10^3 MPa	Coefficient of linear thermal expansion, α_t x 10^{-6} per ^oC	Typical movement, mm per m for 60^oC	Coefficient of Moisture expansion, α_m (% elongation per % change in m.c.)
Cement based composites				
cement mortar	20-35	10-13	0.78	
concrete (normal)	4700 f'c	10-14	0.84	0.003-0.01
cellulose cement sheet	14-26	8-12	0.72	0.0075-0.01
GRC	20-34	7-12	0.72	0.006-0.01
Masonry				
concrete block	15	6-12	0.72	
bricks (clay)	20-34	5-8	0.48	
Metals				
cast iron	137	10	0.60	
mild steel	210	12	0.72	
stainless steel	200	18	1.1	
aluminium alloys	73	24	1.4	
copper	95-130	17	1.0	
bronze	100	20	1.2	
zinc	140-200	23-33	1.4-2.0	
lead	14	30	1.8	
Building stone				
granite	20-60	8-10	0.50	
limestone	10-80	3-4	0.24	
marble	35	4-6	0.36	
slate	10-35	9-11	0.66	
Plastics				
acrylic	2.5-3.3	50-90	3.0-5.4	
GRP	6-12	20-35	1.2-2.1	
polycarbonate	2.2-2.5	60-70	3.6-4.2	
PVC	2.1-3.5	40-70	2.4-4.2	
glass (plain-tinted)	70	9-11	0.66	
Timber				
Pinus radiata	9.0 Dry 5.8 Green	3 with grain 30 across grain	0.18	(below fibre saturation point) 0.12 radial 0.22 tangential 0.005 longitudinal
particleboard *	3-6 ~~6-10~~	30-40		0.5-1.5 thickness 0.02-0.03 in-plane
plywood *	6-12	20		0.11 thickness 0.01 in-plane
hardboard *	[illegible] 4-~~6~~	10	0.6	0.01-0.02 [illegible]

* Modulus of rigidity (G)=E/15

Table 7: Estimated extreme temperatures on buildings

Building Element	Extreme Temperatures °C	
	maximum	minimum
Precast concrete; light-coloured masonry wall (outer 75mm); exposed concrete eaves; edges of floor slab	50	-10
Similar construction, but dark coloured	65	-10
Black glass; ceramic tiles, or metal insulated behind	80	-15
White glass; ceramic tiles; or metal insulated behind	60	-15
Black metal panel, exposed behind clear glass and insulated behind	130	-5
Clear glass in front of dark insulated background such as panel above	80	-15
Aluminium mullion in a curtain wall (natural colour or white)	50	-5

Thermal movement can be calculated from

$$\Delta l_t = \alpha_t . l . \Delta T$$

where Δl_t = change in length due to temperature variation (m)

α_t = coefficient of linear thermal expansion (per ^oC)

l = original dimension of component (m)

ΔT = change in temperature (^oC)

(Refer to Table 6 column 3 for values of α_t)

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

Example: A plastic canopy was installed over a mining conveyor in Australia. The structure was assembled on the ground and lifted onto the supporting pole structure. The canopy failed during winter at the fixing points where insufficient provision for movement had been allowed in the design. The actual expansion had been greater during the summer, where very high temperatures had been experienced. However the canopy remained ductile at these elevated temperatures and could accommodate deformation around the fixing points. During the low winter temperatures, the canopy became brittle and fractured at the fixings.

2.5.2 Moisture Movements

Moisture movements result from porous materials absorbing and releasing moisture. Some moisture related movement is irreversible, and other reversible.

Irreversible changes are usually those that occur shortly after manufacture, commonly known as drying shrinkage. Examples of this action are most readily seen in concrete or cement-based products. About 50% of the total drying shrinkage occurs within 3 months of manufacture. Drying shrinkage also occurs in many timber components (despite seasoning).

The irreversible expansion of bricks and tiles that has been experienced in the UK and Australia is not normally found in the same items manufactured in New Zealand. It is always a possibility with imported ceramic products, and means that it is important to include provision for movement in such building components. (See e.g., BS 5385).

Reversible moisture movements occur in some materials which are hygroscopic. Such materials absorb and release moisture, usually initiated by changes in relative humidity. 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 30% by weight. Above FSP, moisture movement 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 Pinus radiata the tangential shrinkage is 0.22% and the radial shrinkage is 0.12% per percent moisture change. 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 Pinus radiata has a moisture content in the order of 25% and is likely to experience mould growth, nail rot and other deterioration characteristics. Typical timber moisture contents more commonly encountered are well maintained mechanically heated internal swimming pool (20%); normal residential framing (10-12%); and air conditioned offices (5-7%).

Moisture movement can be calculated from:

$$\Delta l_m = \alpha_m . l . \Delta m$$

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)

Δm = change in moisture (%)

(Refer to Table 6 column 5 for values of α_m)

2.6 Long Term Deflection: Creep

2.6.1 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 is considered to contribute to the creep within the timber.

The effect of creep is accounted for in 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 the Timber Design Code (Bibliography 1.6) 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 between cells) 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 by up to 50%.

2.6.2 Creep in Reinforced Concrete

The mechanism of creep in concrete is also related to the change in 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) the Concrete Design Code (Bibliography 1.1) suggests that creep and shrinkage be accounted for as a simple factor $Kcp=(2-1.2(A's/As))> 0.6$. This factor is applied to the deflection due to dead load and sustained live load. The Code recognised that studies showed creep may vary by up to 400% depending on the aggregate used.

Dynamic Effects

3. Problems with Vibration

Building vibration problems can be classified as being caused either by continuous or transient vibrations.

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 are synchronised with one of the structure's natural frequencies of vibration, resonance may occur (see section 3.2). This could greatly magnify the effects of the vibration to the extent that structural damage, overloading, and fatigue may result. Such behaviour is not affected by the damping of the structure. The response can be very severe. It is usual, where practical, to isolate the source of the vibration from the building by incorporating suitable base isolating and damping devices thus avoiding the transmission of the excitation to the building. Many continuous vibration problems can be treated in this manner. Where such isolation is not practical, it is important to ensure that the frequency of the forcing function does not coincide with the natural frequency of either the building or the supporting components.

Transient vibrations are caused by intermittent excitations, such as footsteps or other impulses. The motion is significantly influenced by the degree of damping of the supporting component, and by the effects of higher harmonics of the motion. Transient vibration problems are usually annoying rather than structurally damaging (i.e., rattling of china, felt motion, or rocking of tall furniture). The magnitude of vibration is often magnified by the item which houses the component. The magnitude of damping is dependent on the structural form, the material and items present in the excited area, the distribution of these items, the presence of people and other variables. It is consequently not possible to provide detailed design criteria to preclude all transient vibrations. The static flexibility of the component is indicative of its sensitivity to vibration and guidelines are given in the tables of limiting static deflection to avoid transient vibration problems.

3.1 Acceptable Vibration Acceleration

The level at which vibrations cause problems is very dependent on the person exposed to the vibrations and the circumstances that apply at the time when the vibration occurs. The determination of 'annoying vibrations' has been the subject of many studies but is still indistinct because the sensitivity of each individual to vibration varies significantly, influenced by external environmental factors which are themselves variable and difficult to consistently reproduce in laboratory conditions.

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' has determined acceleration levels of 'equal annoyance' (the 'base curve'). Two such curves are available, the first for acceleration in the foot-to-head direction and the other in the front-to-back direction. In both cases the acceleration level varies with the frequency of vibration. Multiplying factors are applied to the base curves to determine nominally acceptable levels of acceleration which should avoid vibration problems.

The multiplying factors consider the various environmental factors which influence the response of experimental subjects in the assessment of 'acceptable acceleration limits'. The factors have been identified to include the following:

surrounding environment	Tranquil or active surroundings (e.g., home, office or gymnasium)
frequency of vibration	higher frequency accelerations are noticed less
duration of vibration	short duration vibrations with higher accelerations are more tolerable
direction of motion	Foot-to-head acceleration is more annoying than front-to-back
expectation	events which are forewarned are more acceptable
timing of vibration	Motion at night is more annoying than the same motion during the day

The combination of these factors has resulted in the curves shown in Figure 4 on page 21. 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. Front-to-back vibrations are given in BS6472:1984 (Bibliography 1.9) and have a different frequency response spectra.

3.2 Structural Stability and Resonance

When the excitation frequency coincides with one of the fundamental frequencies of the supporting element, resonance occurs. During this state, the motion generated may be magnified by up to 20 times that of the static load condition. Acceleration, velocity, and displacement of the surface motion are all significantly magnified.

Traditional, relatively stiff, building components will have a natural frequency of vibration greater than 20 Hz. Resonant behaviour will only result if the forcing function has a similarly high frequency. Oscillating machinery could initiate this action but would usually be isolated from the structure to minimise the effect. A cyclic forcing function that cannot be easily isolated is the rythmic behaviour of coordinated human activities such as dancing, gymnastics etc. Overseas studies confirm that crowds can be synchronised, by music or other means, up to frequencies of 6 Hz, beyond which they become discoordinated and a random forcing function results. Modern materials and building techniques may result in some floor systems having a natural frequency at or lower than 6 Hz. Such flows should be checked for resonance if they are supporting assembly occupancies. A design method for such a check is detailed in section 3.3.

The design procedure (see 3.3) to avoid resonance and potential structural damage which may result, does not necessarily avoid transient vibration problems.

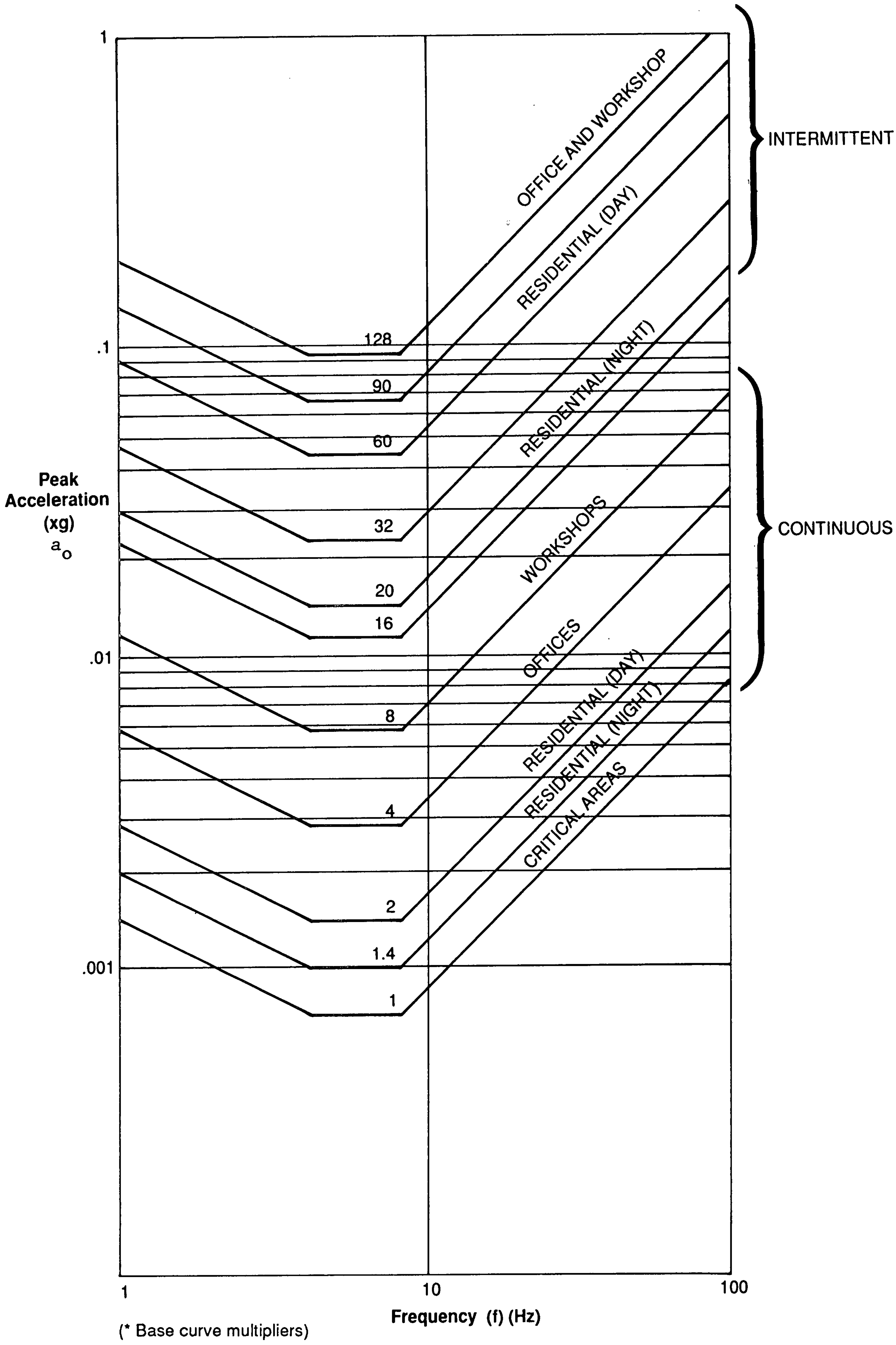


Figure 4 : Acceleration Curves - Foot-to-head direction. (From BS 6472 : 1984)

3.3 Avoidance of Resonance

The following procedure will identify floor systems which are likely to resonate when subjected to cyclic excitations. To avoid this phenomenon, the component may need to be stiffened, increasing its natural frequency beyond that of the exciting forces.

The procedure is specifically for buildings which are subjected to periodic excitation generated by human activities, but can also be applied to other forms of excitation. The levels of acceptable acceleration outlined in Table 9 or Figure 4 may be used in item 6 of the design procedure outlined below. Transient vibrations, and their prevention, cannot be resolved using the methods outlined since the effects of damping and higher order vibrations are not considered during resonant response.

It may be necessary to check more than one load case and more than one type of activity.

The procedure:

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 to Table 8 for guidance.

2. Select an appropriate forcing frequency, f, and dynamic load factor (DLF) which is applied to Wp to determine the dynamic load Wd (refer Table 8). For jumping exercises, the frequency for both the first and second harmonic (1.5-3 Hz and 3-6 Hz) should be checked.

3. Select an acceptable limiting acceleration, a_o, at the centre of the floor (refer Table 9 and Figure 4 for guidance).

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

5. Determine the fundamental frequency, fo, for the floor structure. (include self weight plus actual applied live load)

6. Check that the following condition is satisfied:

$$fo > f\sqrt{1 + \frac{K.Wd}{a_o.Wt}}$$

where

fo	=	the fundamental frequency of the supporting element (Hz)
f	=	the forcing excitation frequency (Hz)
Wp	=	weight of participants (kPa)
Wd	=	the dynamic load component (kPa)
Wt	=	the total load (kPa)
a_o	=	limit of 'acceptable' acceleration as a ratio of g
g	=	gravitational acceleration (9.81 m/sec^2)
K	=	1.3 for simply supported beams
	=	1.3 for fixed end supported beams
	=	1.4 rectangular plate (a/b=2)
	=	1.5 for cantilevers
	=	1.6 for a square plate (simply supported or fixed)

If the criteria are not satisfied, the acceptable level of acceleration will be exceeded. Options available 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 acceptance criteria have been derived from the equations of motion for the element, with substitution of the static deflection due to the dynamic load component Wp and the peak acceleration assuming sinusoidal frequency excitation. Damping is ignored as is appropriate with the component nearing resonance.

3 4. Estimation of Dynamic Load Parameters

Determine the frequency of excitation to which the floor may be subjected. The frequency of excitation impulses generated by human activities is in the range of 1 to 4 Hz. The frequency of many activities that are likely to generate such vibrations, and the distribution of live load on such occasions are assessed in Table 8.

The dynamic load factor (DLF), 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 9). These are based on a minimum of 20 people and may be higher for fewer, better co-ordinated people. A probable distributed weight of participants to be used in conjunction with the dynamic load factor is given.

Table 8: Estimated dynamic loading intensity

Activity	Forcing Frequency f, (Hz)	Weight of Participants Wp, (kPa)	Dynamic Load Factor DLF	Dynamic Load Wd, (kPa)
Walking	1.5 to 2.5	0 .8 ($1m^2$/person)	0.6	0.5
Jogging	2.5 to 3	0.6 ($1.2m^2$/person)	0.4	0.25
Dancing	1.5 to 3	0.6 ($2.5m^2$/couple)	0.5	0.3
Lively concerts or sports events	1.5 to 3	1.5 ($0.5m^2$/person)	0.25	0.4
Jumping Exercises	1.5 to 3	0.4 ($2m^2$/person)	1.5	0.6
	3 to 6	0.4 ($2m^2$/person)	0.25	0.1

Table 9: Application of Vibration Criteria to different Activities and Floor Constructions

Type of floor	Dead load, (kPa)	Forcing Frequency f, (Hz)	Weight of Participants Wp, (kPa)	Dynamic Load Wd, (kPa)	Total Weight Wt, (kPa)	Minimum fundamental frequency fo, Hz
	Dancing and dining, $a_o = 0.02$ g					
concrete	5	3	0.6	0.3	5.6	6.4
steel joist	2.5	3	0.6	0.3	3.1	8.1
timber	0.7	3	0.6	0.3	1.3	12.0
	Lively concert or sports event, $a_o = 0.05$ g					
concrete	5	3	1.5	0.4	6.5	4.8
steel joist	2.5	3	1.5	0.4	4.0	5.7
timber	0.7	3	1.5	0.4	2.2	7.2
	Jumping exercises, $a_o = 0.05$ g					
concrete	5	6	0.4	0.1	5.4	7.3
steel joist	2.5	6	0.4	0.1	2.9	8.3
timber	0.7	3	0.4	0.6	1.1	11.7

(Note: Table 9 compiled using Eq 6 from Section 3.3 using K=1.3 for beam elements.)

3.5 Fundamental Frequency of Building Components.

The fundamental frequency of vibration for simple beams, continuous beams, fixed ended beams and cantilevers can be approximated as

$$fo = \frac{1}{2\sqrt{ds}}$$

where ds is the static midspan deflection (mm) due to the dead load and sustained portion of the live load (include the weight of the participants, furniture, etc.).

This approximation is generally adequate for design purposes and is independent of the loading or fixity of the component.

3.6 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 (3-12 Hz), rather than taller buildings (0.2-1 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 (3.8-7 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.

BIBLIOGRAPHY

Part 1. Related Building Codes and Standards

1.1 Standards Association of New Zealand. Code of practice for the design of concrete structures. NZS 3101: Parts 1 and 2: 1982. Wellington.

1.2 Standards Association of Australia. SAA steel structures code AS 1250 - 1981. North Sydney

1.3 British Standards Institution. Code of practice for the structural use of concrete. BS CP 110: 1972. London.

1.4 Canadian Standards Association. Steel structures for buildings. CSA S16: 1969. Rexdale, Ontario

1.5 Canadian Standards Association. Code for engineering design in wood. CAN-086-M80: 1980. Rexdale, Ontario.

1.6 Standards Association of New Zealand. Code of practice for timber design. NZS 3603: 1981. Wellington.

1.7 Standards Association of New Zealand. Code of practice for light timber frame buildings not requiring specific design. NZS 3604: 1984. Wellington.

1.8 Standards Association of New Zealand. Code of practice for general structural design and design loadings for buildings. NZS 4203: 1984. Wellington.

1.9 British Standards Institution. Evaluation of human exposure to vibration in buildings (1 Hz to 80 Hz). BS 6472:1984. London.

1.10 International Organisation for Standards. Guidelines for the evaluation of the response of occupants of fixed structures, especially buildings and off-shore structures, to low-frequency horizontal motion (0.063-1 Hz) 1984-08-15. ISO 6897: 1984. Switzerland.

1.11 National Research Council of Canada: 1985 Supplement to the National Building Code of Canada. Associate Committee on the National Building Code, Ottawa, Ontario.

Part 2: Related General Papers

2.1 Canada Institute for Scientific and Technical Information, 1980. Deformation Requirements for Building. Ottawa. (translation of: Vervormingseisen voor bouwconstructies. The Steelbuilding Association, Rotterdam and the Concrete Association, Zoetermeer, 1969).

2.2 Pfrang, E.O. and Yokel, F.Y. 1969. Structural Performance Evaluation of a Building System. National Bureau of Standards. Washington.

2.3 Galambas, T.V. et al. 1973. Structural Deflections: A Literature and State-of-the-Art Survey. National Bureau of Standards. Washington.

2.4 Alexander, S.J. and Lawson, R.M. 1981. Design for Movement in Buildings. Construction Industry Research and Information Association (CIRIA) (Technical Note 107). London.

2.5 Blakey, F.A. and Heiman, J.L. 1974. Transverse Deflection of Floors, Creep, Shrinkage and Temperature Effects. Division of Building Research, CSIRO (Report 29). Melbourne.

2.6 Tilly, G.P. et al. 1984. Dynamic behaviour of footbridges. IABSE Surveys 5-26 /84. International Association for Bridge and Structural Engineering. Zurich.

2.7 Building Research Establishment. 1983. Vibrations: building and human response. BRE Digest 278, Watford

2.8 National Research Council of Canada. 1984. Vibrations in buildings. Canadian Building Digest CBD232, Division of Building Research, Ottawa.

2 9 Phillips, M.H. et al. 1984. Horizontal vibration of houses. Report No 5 - 84 /3. Ministry of Works and Development, Central Laboratories, Lower Hutt.

2.10 Building Research Establishment Information Paper. 1983. Determining the probable dynamic response of suspended floors. Information Paper IP 17/83, Watford.

2.11 Canada Institute for Scientific and Technical Information. 1984. Vibrations of building structures caused by human activities, case study of a gymnasium. (translation from: Schweizer Ingenieur and Architect, 101(6): 104-110, 1983). Ottawa.

2.12 Rainer, J.H. 1984. Dynamic Response of a Gymnasium Floor. Division of Building Research, Research Note No 213, National Research Council of Canada. Ottawa.

2.13 Allen, D.E. et al, 1985. Vibration Criteria for Assembly Occupancies. Division of Building Research, Paper No 1310 , National Research Council of Canada. Ottawa.

2.14 Steffens, R.J. 1974. Structural Vibration and damage. British Research Establishment Report. Watford.

2.15 Rainer, J.H. et al, 1986. Dynamic Loading and Response of footbridge. National Research Council of Canada. Ottawa.

2.16 Crist, R.A. and Shaver, J.R. 1976. Deflection performance criteria for floors. U S National Bureau of Standards. Washington.

2.17 Ellingwood, B. and Tallin, A. 1984. Structural Serviceability floor vibrations. Journal of Structural Engineering.

2.18 Rainer, J.H. 1986. An Approach to a Vibration Standard for Buildings. Institute for Research in Construction. Paper No.1387. National Research Council of Canada. Ottawa.

Part 3. Deflection of Concrete Structures

3.1 Fling, R.S. et al. (Subcommittee , ACI Committee). 1968. Allowable Deflections. (Title No. 65-31). American Concrete Institute Journal. Detroit, Michigan.

3.2 American Concrete Institute. 1977. Building Code Requirements for Reinforced Concrete (and Commentary). (ACI 318-77) Detroit, Michigan.

3.3 American Concrete Institute. 1974. Deflection of concrete structures. (ACI publication SP43) . Detroit, Michigan.

3.4 Base, S.C.C. et al. 1972. Handbook on the Unified Code for Structural Concrete (CP 110). Cement and Concrete Association. London.

3.5 MacGinley, T.J. 1978. Reinforced concrete: Design, Theory and Examples. Spur. London.

3.6 Park, R. and Paulay, T. 1975. Reinforced Concrete Structures. Wiley. New York.

3.7 Walsh, P.F. 1975. Reinforced concrete deflection design. CSIRO. (Report 39). Division of Building Research, Melbourne.

3.8 Allen D.E. Rainer J.H. and Pernica G. 1977. Vibration criteria for long span concrete floors. Division of Building Research. National Research Council of Canada. Ottawa.

3.9 Paulay, T. and Williams, R. 1980. Reinforced concrete ductile shear walls Vol 13 No 2 Bulletin of the New Zealand National Society of Earthquake Engineering, Wellington.

Part 4: Deflection of Steel Structures

4.1 New Zealand Steel Ltd, the Profile Manufacturers' Association & Building Research Association of New Zealand. 1981. Profiled Metal Roofing Design and Installation Handbook: prepared by the Corrugated Steel Manufacturers Association. Probe Publications. Auckland.

4.2 Fletcher-Brownbuilt Company. 1981. Steel Products Trade Catalogue. Penrose Associates. Auckland.

4.3 Douhan, L. 1980. Deformation Limits for Roofs and Walls of Profiled Sheeting. Swedish Council for Building Research (Document D32). Stockholm.

4.4 American Institute of Steel Construction. 1978. Specification for the Design Fabrication and Erection of Structural Steel of Buildings. New York.

4.5 Lay, M.G. 1974. Source book for the Australian Steel Structures Code AS 1250. Sydney. Australian Institute of Steel Construction.

4.6 Constructional Steel Research and Development Organisation (Constrado). 1983 Steel Designers Manual. Fourth Edition (Revised), Suffolk.

Part 5: Deflection of Timber Structures

5.1 Pearson, R.C. Kloot, N.H. and Boyd J.D. 1962. Timber Engineering Design Handbook. Melbourne. Jacaranda Press.

5.2 Henderson and Pollard Industries Limited. 1976. Plywood Products Trade Catalogue. Auckland.

5.3 Whale, L. Vibration of Timber floors - a literature review. 1983. Timber Research and Development Association, Research Report 2/83. High Wycombe, Bucks.

5.4 Atherton, G.N. Polensek, A. and Corder, S.E. 1976 Human response to walking and impact vibration of wood floors. Forest Products Journal. Vol. 26 No. 10, October.

Appendix A: Suggested Maximum Span Depth Ratios for Beams

Building Material	Member or Grade	Simply Supported	Both Ends Continuous	Cantilever
Steel	(NZS 3404) fy = 250 MPa fy = 340 MPa	 21 16		
Concrete	(NZS 3101) One way slab fy = 275 MPa	 25	 35	 13
	fy = 380 MPa	21	29	11
	Beams or ribbed one way slabs fy = 275 MPa fy = 380 MPa	 20 17	 26 22	 10 8
Timber	(NZS 3603) (NZS 3615)		15	G=E/16
	Note: Because of the low shear modulus of timber, those beams which are relatively deep (span/depth < 10), shear deflection can contribute more than 20 per cent additional deflection			

APPENDIX B: Suggested Cambers

1. Glued Laminated Beams:

   1/400th of the span or the long term deflection due to dead load plus the sustained portion of the live load (D+Ls) (i.e. use E/1.5 as per clause 8.7.4.1 of NZS 3603).

2. Nail Plate Trusses:

   a) Pitched trusses 1.5 mm per metre of span
   b) Parallel chord trusses 2.0 mm per metre of span.

3. Reinforced Concrete Beams:

   It is suggested that the camber should equal the long term deflection due to dead load plus the sustained portion of the live load (D+Ls).

4. Prestressed Concrete Beams:

   It is good practice , whenever possible, to balance the deflection resulting from dead load by the camber produced by the prestress.

   Frequently, a designer can put slightly more camber into the beam, so that flexural creep tending to camber the beam upwards will just balance the downwards deflection resulting from loss of prestress.

APPENDIX C: Notation

α_m Moisture expansion coefficient

α_t Thermal expansion coefficient

A Area (m^2)

A's Concrete compression reinforcing steel Area (mm^2)

As Concrete tension reinforcing steel Area (mm^2)

a_o Limit of 'acceptable' acceleration as a ratio of g

D Dead load (kN)

d Deflection (mm)

d(add) Deflection under additional loads, i.e., the additional deflection occurring after installation of non-structural elements

Note:

1) With steel members, d(add) is simply the deflection due to the sustained live load
2) With members subject to creep, d(add) is the total long term deflection under D + Ls, minus the initial deflection due to dead load only.

d(dyn) Maximum deflection at any arbitrary point of the beam subjected to a periodic load

d(stat) Immediate deflection due to static load (usually the maximum design load)

DLF Dynamic load factor

E Modulus of elasticity (MPa)

Eq Earthquake load (kN)

Ec Modulus of elasticity of concrete = 4700 f'c (MPa)

e Strain

f Frequency of applied load (Hz)

fo Fundamental frequency of building component (Hz)

fy Steel yield stress (MPa)

f'c Concrete compressive stress (MPa)

h	Depth, thickness or height (m)
I	Moment of inertia of a Gross Section (m^4)
K_2	Timber creep allowance factor
Kcp	Concrete creep and shrinkage factor
l	length or span (m)
Δl_m	Change of length due to moisture change (m)
Δl_t	Change of length due to temperature change (m)
L	Live load (kN)
Lc	Concentrated live load (usually 1 kN)
Ldyn	Dynamic live load (kN)
Ls	Sustained live load - that portion of live load which is essentially permanent and which can give creep deflections or can affect dynamic response (kN)
M	Bending moment
sag	d_o - camber
S	Snow load
t	Time
ΔT	Change in temperature (Degree C)
w	Load on beam per unit length (kN/m)
W	Wind load
Wd	Dynamic load component (kPa)
Wp	Weight of participants for dynamic excitation (kPa)
Wt	Total active weight for dynamic behavior (kPa)
Z	Modular ratio (m^3)

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