STUDY REPORT

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A LITERATURE SURVEY OF DESIGN METHODS FOR FLOORS SUBJECTED TO OCCUPANT-INDUCED VIBRATIONS

K.Y.S. Lim
PREFACE

This literature survey forms the first part of a research programme being undertaken by BRANZ to prepare design information on occupant-induced vibrations associated with methods of floor construction used in New Zealand.

ACKNOWLEDGEMENTS

The author acknowledges the help of Dr David Allen and Professor Tom Murray in preparing this report.

AUDIENCE

This report is intended for design engineers and other workers in the field of structural engineering research.
A LITERATURE SURVEY OF DESIGN METHODS FOR FLOORS SUBJECTED TO OCCUPANT-INDUCED VIBRATIONS

BRANZ Study Report SR 31

K.Y.S. Lim

REFERENCE


KEYWORDS

From Construction Industry Thesaurus - BRANZ Edition: Acceleration; Acceptability Limits; Continuous Vibrations; Criteria; Damping; Design; Displacement; Dynamics; Floors; Frequency; Literature Survey; Occupant; Remedial Measures; Resonance; Static; Stiffness; Structure; Transient Vibrations, Walking.

ABSTRACT

The use of high-strength building components and the utilisation of long-span composite constructions lead to noticeable floor vibrations in buildings. The desire of owners to have open plan buildings, and the changed usage of buildings from original designs, also contribute to the problems of unacceptable vibrations.

This work forms the first part of a research programme being undertaken by BRANZ. In this report, a literature survey of the available design methods, acceleration acceptability limits, and remedial measures for occupant-induced floor vibrations has been carried out. Some design recommendations are given.
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1.0 INTRODUCTION

Over the past 15 years, a great deal of research effort has been focused overseas on the dynamic characteristics of long-span steel joist-concrete deck floor systems. These systems are generally lighter and more flexible than their predecessors (Pernica, 1987). They possess less inherent damping (Allen and Rainer, 1975) and are thus more susceptible to vibration problems.

Vibrations in buildings are also becoming more noticeable because of lighter structures due to the higher strength and more efficient use of materials. The desire of owners to have open plan buildings with larger areas free from columns and partitions has also contributed to vibration-related problems. Problems also result from the different usage of buildings from what they were originally intended and designed for.

Current practices within the New Zealand construction industry are following overseas trends to minimise construction costs by using high strength building components and long-span composite floors, resulting in more flexible structures. Vibration problems are currently infrequent in New Zealand buildings. This situation was similar overseas until a decade or so ago (Huggins and Barber, 1982; Allen et al, 1987). Since then such problems have became more common and it may be expected that New Zealand will follow this trend unless designers are more aware and can design for their exclusion.

The objective of this survey was to identify and discuss the alternative design methods, acceptability limits and remedial measures available to address the problem of occupant-induced vibrations. Design methods for other sources of vibrations are outside the scope of this work. It should be noted however, that movements generated by rare events such as very strong wind storms or earthquakes are usually tolerable provided the frequency of occurrence is low (Cooney and King, 1988). Construction activities, which may be intense over a relatively short time frame can also be considered within this category.

2.0 LITERATURE SURVEY

The literature relating to occupant-induced floor vibrations which were considered relevant have been reviewed in this study. Brief descriptions of the various methods, acceptability limits and remedial measures are given in the sections to follow. Other materials recommended for further reading include:

Reiher and Meister (1931); Lenzen (1963); Megard et al (1971); Wiss and Parmelee (1974); Crist and Shaver (1976); Jeary (1983); Whale (1983); Chien and Ritchie (1984); Ellingwood and Tallin (1984); Ericksson and Ohlsson (1988); Clifton (1989).

2.1 Types of Vibrations

The vibration of a floor may refer to either a steady-state harmonic vibratory motion (continuous) which lasts for several minutes, or a rapidly decaying transient motion induced by impulse loading which is usually over in a fraction of a second.
Continuous vibrations arise from periodic forces which continue for a significant period of time (e.g. certain rhythmic human activities such as dancing). Such excitations may induce resonance in the floor when the frequency of the periodic forces coincides with one of the natural frequencies of vibration of the floor. Resonant response greatly magnifies the effects of the vibration (by up to 20 times that of the static condition) to the extent that it may result in structural damage or, in the extreme, collapse. Recently, it has been recognised that resonance may also occur when the higher harmonics of the excitation frequency coincides with any of the fundamental frequencies of the floor (Allen et al, 1985; Bachmann, 1988; International Standards Organisation (ISO), 1989a). The levels of response at resonance generally decrease with increasing damping, although the presence of the damping does not cause the response to decay.

Transient vibrations are caused by intermittent excitations, a single impulse, or a series of such impulses imparted to the floor (e.g. humans walking across the floor). Lenzen (1962) considered transient vibration to be a single pulse which fits the vibration produced by a single footfall or someone jumping. Transient vibration problems are usually annoying rather than structurally damaging (e.g. felt motion or rattling of china). The response of a floor to such excitations is significantly influenced by the damping within the floor and within the excited masses. The magnitude of the damping is dependent on the material and method of construction; floor mass, partitions, support conditions, the presence of people and other variables.

Walking is the main cause of transient vibrations in floors of offices and residential buildings. In commercial environments such as shopping centres, walking vibrations are, however, considered to be of a continuous nature. Floors in these environments should be checked for resonant response resulting from the higher harmonics, particularly the third, and in more recent times the second harmonic (Allen, 1990).

Ohlsson (1984) made a distinction between annoying vibrations and springiness. Annoying vibrations are vibrations caused by somebody other than the person who is disturbed by them while springiness is a disturbing sensation due to a deflection and vibration of the floor at the point of load application.

2.2 Types of Floors

The literature revealed that available design methods to counter the problems of floor vibrations can be distinguished by reference to the floor construction (i.e. light or heavy-weight). Some of the design methods are applicable to both types of floors (Ellingwood and Tallin, 1984; Allen et al, 1985; Allen, 1990).

2.2.1 Light floors

For the purposes of this study, light floors are considered to be those constructed with timber or composite light-weight joists (Hveem, 1987) and a timber deck. They are predominantly used in residential or light
industrial buildings (Figure 1a), and their fundamental natural frequency ($f_o$) is normally greater than 8 Hz. Such floors usually span less than 6 m. Buildings constructed to the New Zealand light timber frame building code, NZS 3604 (Standards Association of New Zealand (SANZ), 1984a) are light-weight floors.

These floors do not normally experience resonance response resulting from vibrations (Allen et al, 1987). This is because the natural frequencies of hung-parts of the human body (lung, heart, etc.) lie in the region of 5 to 10 Hz (Harris and Schoenberger, 1965). Resonance of the human body that can experience annoyance is therefore avoided. Walking has an excitation frequency of up to 2 Hz, and it would require at least a fourth harmonic of the excitation frequency to cause resonance in such floors. Ohlsson (1984) however, recommended that a check for continuous vibrations be carried out when the primary floor support spans more than 4 m, in occupancies with an intensive movement of people (such as in a shopping centre). Although resonance from occupant-induced vibrations is unlikely when the fundamental natural frequency of the floor is greater than 8 Hz, resonant response should be checked for floors with a fundamental frequency of less than 8 Hz.

Polensek (1975) reported on free vibration tests carried out on full size floors and obtained damping ratios (D) of between 4 and 6%. Ohlsson (1984) recommended a damping ratio of 1%, while DP 10137 (ISO, 1989a) recommended a representative value of 2% for bare wood joist floors. Chui and Smith (1989) measured the damping ratios of 1050 mm long with cross section of 50x45 mm specimens under different support conditions and concluded that the support conditions of the specimens significantly influence its damping. These studies indicate that the damping of such floors vary with different methods of construction and support conditions. It may therefore not be appropriate to adopt floor damping values recommended overseas.

A feature of significance for light-weight wooden floors is that they are strongly orthotropic. This leads to dynamic response characterised by closely spaced modes of vibration i.e., the second frequency can be only 15-20% higher than the first, etc. (Chui, 1986; Ohlsson, 1988a). Debate continues whether these higher modes can be ignored for these floor systems.

2.2.2 Heavy floors

Heavy floors are those that are constructed with an insitu concrete topping, i.e., insitu concrete floors, steel joist metal deck or precast concrete floors with an insitu concrete topping. They are predominantly used in commercial high-rise buildings and with the trend towards using longer spans (greater than 6 m) tend to have a fundamental natural frequency ($f_o$) of less than 8 Hz. Thus, both resonant response and transient vibrations need to be considered in the design.

The design methods recommended overseas cannot be readily applied to New Zealand floors because, at present, there is insufficient design information available on their damping characteristics. The method of construction and the code requirements of New Zealand structures may be different to overseas and therefore the damping values recommended overseas may not be appropriate for New Zealand floors.
Over the last few decades, most floors constructed in New Zealand have used precast concrete floors with an insitu concrete topping. The precast floors may be of concrete ribs with timber infill; double tees, dycore or flat slab (Figure 1b). There is currently a trend towards using composite construction; steel beams with a metal deck and an insitu concrete topping (Heavy Engineering Research Association (HERA), 1988), consistent with the preferred practice overseas. The structural resisting system for such buildings normally uses steel frames. NZS 4203 (SANZ, 1984b) has provisions for seismic requirements which inherently lead to more robust structures with greater mass than overseas. The primary structural resisting elements have been constructed in reinforced concrete frames or reinforced concrete shear walls, which are also heavier and more rigid that those using steel frames.

Moreover, the damping values (D) recommended overseas are varied. Clifton (1989) suggested that a finished floor with ceilings and ducts would have a minimum damping value of 4%; 2% for the bare floor, and 1% each from the ceiling and ducts. Indeterminate partitions are those which are fixed to the floor system in at least three locations. It was further claimed that such partitions would add between 10-20% to the floor damping, when they are installed closer than five times the spacing of the secondary floor beams (Also Murray, 1989). Demountable, part-height partitions typically used in open plan offices will provide between 1-2% damping if spread over at least 70% of the appropriate floor area.

Appendix G of CAN3-S16.1 (Canadian Standards Association (CSA), 1984) recommends damping values of 3, 6 and 12% for bare composite floors; finished floors with ceilings, ducts, and finished floors with partitions respectively. Allen (1990) however recommends the damping values of 1.5, 3.0 and 4.5% for the above three stages when considering resonant response. He indicated that vibration decay from heel impact includes a component for geometric dispersion of vibration, as well as frictional and material damping ratios. More recent testing and modal analysis showed the frictional and material damping values to be half that obtained from heel drop impact tests. Wyatt (1989) also recommended damping values similar to that of Allen.

Only the first mode response of heavy floors need generally be considered. Higher modes of heavy floors die out quickly and do not cause discomfort (Allen and Rainer, 1975). Higher modes also have very small amplitudes and higher damping (Allen and Swallow, 1975). The fundamental natural frequencies of heavy floors are close to the excitation frequency caused by common human activities, and therefore needed to be considered (Ellingwood and Tallin, 1984).

2.3 Acceptability Limits

Two main measures of acceptability criteria for occupant-induced floor vibrations were uncovered during the review. These were limiting the acceleration or limiting the velocity response.

Ellingwood and Tallin (1984) stated that although limiting the velocity of vibration of a floor is acceptable, limiting the acceleration of vibration of a floor is a better indication of perceived motion and tolerance because the limit of acceptable acceleration is relatively constant over the 1 to 10 Hz frequency range. Ohlsson (1988a) suggested that for floors
with natural frequency of greater than 8 Hz, the vibration velocity is a better measure of performance while the vibration acceleration is more suitable for long-span floors where the fundamental frequency is below 8 Hz.

Smith (1990) noted that for continuous vibrations, it is not important whether a peak velocity or peak acceleration criterion is used to judge acceptability limit. For transient vibrations in floors with fundamental frequency greater than 8 Hz however, he recommends using the frequency weighted root-mean-square (rms) acceleration limit because of its sensitivity to all major variables influencing acceptability (amplitude, velocity, rate of decay).

In addition to the floor frequency, the choice of the acceptability limit is therefore also dependent on the nature of the excitation. It is therefore inconclusive which is the most effective parameter to identify vibration limits for light floors with frequency greater than 8 Hz. For heavy floors with frequency less than 8 Hz, the acceleration limitation is generally used.

It should be noted that human acceptance of continuous vibrations is at a much lower threshold than that of transient vibrations because of the continuous nature and of the longer duration of the former (Allen and Rainer, 1976).

Likewise, the tolerance level is higher for impact vibrations than vibrations from walking (Whale, 1983). Moreover, the tolerance level is different for each individual, where they are, what they are doing, and whether they are standing, sitting or lying down (Reiher-Meister, 1931). This implies that acceptability limits can be classified under type of occupancy of a building and the type of vibration (continuous, transient).

Becker (1980) recommended that acceptability limits in residential occupancy should be based on local dwelling habits and thus would differ from country to country. This may explain the different tolerance limits obtained by different researchers in different countries (Reiher-Meister, 1931; Lenzen, 1963; Wiss-Parmelee, 1974).

Murray (1989) reviewed various scales for determining the acceptability of floor systems subject to occupant-induced floor vibrations. The scales were shown to be inconsistent and, in general, to underestimate the strong dependence of acceptability on damping. He proposed a criterion which is strongly dependent on damping, as discussed in section 2.6 below.

It has been claimed that the acceptability limits of vibrations determined by means of controlled laboratory studies with volunteers differ from the limits of acceptability in a building where the occupant is not cued to expect movement (Ad Hoc Committee, 1986).

The following section describes briefly the acceleration limits specified in various codes.

BS 6472 (British Standards Institution (BSI), 1984), plots basic acceptability relationships between root-mean-square accelerations and frequencies. It specifies the performance limits with different multiplying factors for different times of exposure, types of occupancy
and types of vibration. Cooney and King (1988) converted these limits to peak accelerations (Figure 2). The ISO 2631-2 (ISO, 1989b) acceleration limits are similar to that of BS 6472.

The hatched lines of Appendix G of CAN3-S16.1 (CSA, 1984) relate to acceleration limits obtained from heel impact tests, and are used as a semi-empirical criterion for evaluating a floor to transient walking vibrations. The solid line in Figure 3 is the recommended acceleration limit for continuous vibrations. Floors with higher damping values are allowed to have higher acceleration limits under walking vibrations.

Commentary A of the Supplement to the National Building Code of Canada (NBCC, 1985) relates to floor resonance under assembly occupancies, and specifies acceleration limits under different constructions and activities as shown in Table 1. The dynamic load factors given in Table 1 are based on a minimum of 20 people and may be higher for fewer, better co-ordinated people. Commentary A has been redrafted (1990) and its main changes are:

(a) more emphasis on mixed occupancies, which is important for health clubs;

(b) inclusion of shear deformation for determining natural frequency, which is important for joists and trusses;

(c) includes a check for the third harmonic of the excitation frequency. The first, second and third harmonics of the excitation frequency have been assigned frequencies of 3, 5 and 7.5 Hz respectively.

The maximum static deflection under the dynamic load, in column 7 of Table 1 has also been deleted.

Ellingwood and Tallin (1984) tabulates acceleration limits under different occupancies/activities, type of vibration, and damping values (Table 2) for several of the general occupancy classifications in ANSI A58.1 (American National Standards Institute (ANSI), 1982).

2.4 Design Methods for All Floors

2.4.1 Crowd loading

Allen et al (1985) derived an expression for the peak acceleration of a floor subjected to sinusoidal loading as experienced under repetitive coordinated human activities. The method is intended to control the resonant response of the floor only. This method is adopted in Commentary A of NBCC (1985). The procedure is as follows:

(a) Determine the type of activity and estimate the density of occupancy from Table 1. When only a portion of the span is occupied by the activity, the weight of the participants \( W_p \) may be calculated from the uniform loading that produces the same centre span deflection as the actual partial loading;

(b) Choose an appropriate forcing frequency \( f \), dynamic load factor \( \lambda \) and an acceptable limiting acceleration \( a_m \), from Table 1 (depending on the activity);
(c) Estimate the total floor load \( (W_t) \);

(d) Determine the fundamental natural frequency \( (f_o) \) of the floor. Weight of the participants and furniture should be included;

(e) If \( a_o/g \) in equation [1] below is less than the limiting acceleration ratio, \( a_m/g \) then the behaviour of the floor is satisfactory.

\[
a_o/g = K' \alpha W_p/W_t \\
\sqrt{[(f_o/f)^2 - 1]^2 + (2DF_o/f)^2}
\]

where \( a_o \) = peak acceleration of floor;
\( a_m \) = limiting acceleration of floor for a particular activity;
\( D \) = floor damping ratio, % of critical;
\( f \) = forcing frequency;
\( f_o \) = fundamental frequency of floor;
\( g \) = acceleration due to gravity \( = 9.81 \text{ m/s}^2 \);
\( K' \) = factor for different types of construction;
\( = 1.3 \) for simply supported and fixed ended beams
\( = 1.5 \) for fixed cantilevers
\( = 1.6 \) for square plates simply supported or fixed;
\( W_p \) = weight of participants (kpa); and \( W_t \) = total weight of floor (kpa);
\( \alpha \) = dynamic load factor.

The methods of calculating the fundamental natural frequency \( (f_o) \) of different types of floors and support conditions are presented in Appendix A.

2.4.2 Walking

Following similar principles to that used in the previous section, the maximum acceleration resulting from the dynamic response of the mass of a floor

\[
a_o/g = 2\alpha P/W \\
\sqrt{[(f_o/f)^2 - 1]^2 + (2DF_o/f)^2}
\]

where \( a_o \), \( D \), \( f \), \( f_o \), \( g \) are as defined in section 2.4.1 above;
\( \alpha \) = dynamic load factor of the n-th harmonic;
\( P \) = weight of person \( = 0.7 \text{ kN} \);
\( W \) = weight of floor;

Allen (1990) simplified this expression considerably by considering that harmonic resonant occurs, i.e. \( f_o = f \) in equation [2a], thus
where \( a_m \) = limiting acceleration;
\( R_t = 0.5 \) is the reduction factor introduced to take into account that full steady-state resonance is not achieved; when the vibration felt is not at midspan, and when someone steps along the beam rather than up and down at midspan.

The frequencies and dynamic load factors of walking are given as:
1. \( = 0.5 \) for the first harmonic frequency of 1.5-2.5 Hz;
2. \( = 0.2 \) for the second harmonic frequency of 3.5-5.5 Hz;
3. \( = 0.1 \) for the third harmonic frequency of 5-7 Hz;
4. \( = 0.05 \) for the fourth harmonic frequency of 7-10 Hz.

In the case of a beam system on rigid supports, Allen suggested that \( W \) be approximated by a simply supported panel with span, \( L \) and width \( B \) given by,

\[
B = 2L \frac{D_y}{D_x} \leq L \quad \text{[3]}
\]

where \( D_y \) and \( D_x \) are flexural rigidities per unit width of the deck perpendicular to the floor span, and in the span direction respectively. The estimation for the weight (\( W \)) of a two-way simply supported system is also given.

Table 3a shows the minimum values of the right hand side of equation [2b] for satisfactory performance of the floor. The limiting \( a_m/g \) value incorporated in the right hand side of equation [2b] is derived by Allen, based on ISO 2631-2 (ISO, 1989b), and resulted in an \( a_m \) of 0.5\% g for the frequency in the 4-8 Hz range. Once the fundamental frequency of the floor has been calculated, the designer checks for the particular harmonic resonant response.

Table 3b shows the damping values recommended by Allen; 1.5, 3.0 and 4.5\% respectively for bare floor; finished floor with ceiling and ducting, and finished floor with full-height partitions.

This method and the method of section 2.4.1 are recommended for use in the design of floor when subjected to resonant or harmonic resonant excitation.

2.5 Design Methods for Light Floors

2.5.1 Static deflection limitation

Hansen (1953) reported on vibration tests carried out in occupied homes and recommended that the static deflection of simply supported floors under a concentrated load of 100 kg should not exceed 0.9 mm to achieve a dissatisfaction rate of less than 6\% and thus satisfy vibration serviceability. Allen et al (1987) noted that resonance from walking
vibration in light floors with frequencies greater than 10 Hz is unlikely. The primary problem was therefore claimed to be footstep deflections which would be best controlled by a static stiffness criterion such as 1 mm maximum deflection under a 1 kN concentrated load. Hveem (1987) noted that there is an increase in the use of new light-weight joists (I-profiles with wood flanges and sheet material web), which are characterised by high stiffness and low weight. He claimed that floors constructed using these joists may span up to 8 m, if based on Hansen’s (1953) criteria. On the other hand, Bier (1989) claimed that to satisfy the static stiffness limit of 0.9 mm under a 1 kN point load, the joist span tables of NZS 3604 (SANZ, 1984a) would have to be reduced to 60% of existing values in some cases.

Onysko (1988) proposed a criterion based on the static stiffness. The final results of the survey of floors in houses revealed that the most important variables discriminating between good and poor floors were:

(a) the deflection under concentrated load;
(b) the peak dynamic response to impulse loading;
(c) the damping ratio;
(d) the frequency of vibration; and
(e) the floor span.

It was found that the deflections of floors under a uniform load was a relatively poor discriminator. A criterion using the static deflection under a 1 kN point load was proposed:

\[ d \leq \frac{8}{L^{1.3}} \]  

[4] (Figure 4)

where \( d \) = the vertical deflection at midspan under a 1 kN point load, in mm; 
\( L \) = the span in m.

2.5.2 Impulse under heel drop

One method is to evaluate the impulsive velocity response \( h'_{\text{max}} \) and the damping coefficient \( D' = f_0 D \) and compare that with Figure 5 (Ohlsson, 1984). Ohlsson acknowledged that while the method has produced good results, it was based on the data from a limited number of tests.

\[ h'_{\text{max}} = 4(0.4 + 0.6 N_{40}) \]  

[5]

\[ \text{MBL} + 200 \]

where \( B \) and \( L \) are the width span of floor in m respectively;
\( D \) = floor damping ratio, % of critical;
\( f_0 \) = fundamental floor frequency;
\( M \) = unit weight of floor in kg/m²;
\( N_{40} \) = number of normal modes with natural frequencies < 40 Hz.
The natural frequencies for isotropic plates can be calculated from equation [17] (Appendix A), and the number of modes below 40 Hz can be counted. For orthotropic plates, the number of modes below 40 Hz can be evaluated from tables provided by Ohlsson.

A criterion and method of design against vibration in light-weight wooden floors has been proposed (Chui, 1986; Chui and Smith, 1987; 1988a). In order to obtain a design expression simple enough to be performed by designers, it was assumed that only the fundamental mode of the floor response contributed. This is compensated for by limiting the application of the procedure to floors which have means of enhancing floor stiffness in the across-joists direction. The use of solid blocking between joists and support of edge joists was considered to satisfy this requirement. Laying of floor sheathing with its stronger axis perpendicular to the joist span would also enhance the stiffness in the across-joist direction. (N.B. The stronger axis is normally along the longest dimension of the sheathing material). It should be noted that this construction technique will increase the stiffness by about 10% for particle board floors, but a much more substantial increase can be expected if using plywood floors.

A limit for the frequency-weighted rms acceleration of 0.45 m/s² and a fundamental natural frequency greater than 8 Hz was suggested. Using the Duhamel integral, Chui (1986) derived an expression for estimating the frequency weighted rms acceleration of a wooden floor under a heel drop impact and is given by:

\[ a'_{\text{rms}} = \frac{2000X}{m \pi (f')^2} \]  

\[ f' = \frac{\pi \sqrt{E_j I_j (n-1)}}{2L^2 \left( \ell_s t B + \ell_j A (n-1) + \frac{280}{L} \right)} \]  

\[ m = \frac{L}{2} \left( \ell_s t B + \ell_j A(n-1) + \frac{280}{L} \right) \]

where
- \( A \) = cross-sectional area per joist;
- \( B \) and \( L \) are the width and span of the floor respectively;
- \( E_j \) = joist modules of elasticity;
- \( f' \) = the fundamental natural frequency of the floor with a 70 kg observer at the centre;
- \( I_j \) = second moment of area per joist;
- \( n \) = number of joists;
- \( t \) = sheathing thickness;
- \( \ell_s \) = joist density;
- \( \ell_j \) = sheathing density;
- \( X \) = a factor depending on the value of \( f' \).

Consistent units must be used throughout. Values of \( X \) for different values of \( f' \) is given in Table 4.

Bier (1989) used the design method of Chui and Smith (1987) to evaluate the floor joist tables of NZS 3604 (SANZ, 1984a) using an allowable acceleration of 0.375 m/s². This value was the actual threshold limit for
annoyance that Chui (1986) obtained from testing in the laboratory. Bier's analysis resulted in a recommendation that allowable joist span tables be reduced by either 10 or 20%; 10% for spans with joist depths greater than 200 mm, and 20% for depths lesser than 200 mm. The recommendation has been submitted to SANZ for deliberation. It is understood that the span of floor joists with depths of 150 mm or less and spaced at 600 mm centres have been reduced (SANZ, 1990).

In an attempt to come up with a universal design method for vibration of timber floors, Chui and Smith (1988b) assessed a wide range of floors designed in accordance with the Canadian Standard using their method and methods of Ohlsson (1984) and Onysko (1988). None of the floors could be classified as acceptable under Onysko's criterion, implying a very stringent limit. Chui and Smith's (1987) method was the only one that has the capability of being sensitive to floor size as a variable. It is understood that Chui and Smith's method will be included in future editions of CIB (International Council of Building) code. This method is therefore recommended for use in design.

2.6 Design Methods for Heavy Floors under Walking Vibrations

Allen and Rainer (1976; and Pernica, 1977) developed annoyance criteria for walking vibrations of long-span floors. The criteria was developed for steel beam or steel joist with concrete deck construction for spans greater than 7 m and natural frequencies less than 10 Hz (1976) and modified for concrete floors (1977). It was expressed in terms of the dynamic properties of a floor measured by a heel impact test. The criteria presented are intended for normal 'quiet' human occupancies (i.e. residences, offices and school rooms). This method is adopted by (CSA, 1984). The method involves computing the fundamental natural frequency of the floor, the peak acceleration response of the heel drop and an estimate of the damping. These are compared with the annoyance criteria in Figure 3. The peak acceleration ($a_o$) is estimated by,

$$a_o = 0.9 \left(2\pi f_o\right) \frac{I_m}{M_e} \quad \text{[9]}$$

$$a_o = 60f_o^3/WBL \quad \text{[9b]} \quad \text{(CSA, 1984)}$$

where $I_m = 70$ Ns, heel drop impulse;
$M_e = $ equivalent mass;
$f_o = $ fundamental frequency of floor;
$W = $ the weight of floor plus contents in kpa;
$B = $ width of floor (in m) = 40 t_c$ for rigid supports and is equal to the tributary width (half the supported spans on both sides) for flexible supports (CSA, 1984), where $t_c$ is the equivalent concrete thickness (See Figure 6).

Becker (1980) developed a design evaluation procedure for vibrations of floors under heel drop and treading load. The method assumed that the floor is represented by a one degree of freedom spring-dashpot system. Using the Duhamel Integral, the dynamic magnification factors applicable to the static deflections under the heel drop and treading loads were derived for varying frequencies of vibrations. The procedure involves the calculation of static deflections and floor frequency. Together with the dynamic magnification factors from the derived response spectra; the
maximum dynamic displacement under the heel drop, and tread loading can be calculated and compared with allowable dynamic deflections. The difficulty with this procedure is that New Zealand currently has no dynamic deflection limitations and no guidelines for designers to evaluate the inherent damping of floors.

Murray (1981, 1985, 1989) presented a criterion based on experimental results from 91 composite steel floor systems. The floor system was excited by a person weighing between 170 and 190 lbs (77 and 86 kg) executing a heel drop impact. The initial displacement amplitude and frequency were recorded and data obtained from occupants who rated the floors as either acceptable or unacceptable. The criterion was proposed to be suitable for design office use for evaluating the acceptability of steel beam or joist-concrete slab floor systems subjected to occupant-induced vibrations. The criterion is claimed to be valid for damping between 4-6% critical and conservative for damping above 6%. The viscous damping,

\[
D > 1.38A_o f_o^2 + 2.5 \quad \ldots \quad [10]
\]

\[
A_o = A_{ot}/N_{eff} \quad \ldots \quad [11]
\]

\[
A_{ot} = DLF d_s \quad \ldots \quad [12]
\]

\[
N_{eff} = 2.97 - \frac{S}{17.3t_c} + \frac{L^4}{200EI} \quad \ldots \quad [13]
\]

where \(A_o\) = initial amplitude of the floor system due to a heel drop impact;
\(A_{ot}\) = initial displacement of a single tee beam due to heel drop impact;
\(D\) = floor damping ratio, % of critical;
\(f_o\) = the natural frequency of the floor system, Hz
\(DLF\) = dynamic load factor (Table 5);
\(d_s\) = static deflection due to a 2.67 kN force;
\(N_{eff}\) = number of effective tee beams;
\(S\) = beam spacing greater than 750 mm;
\(t_c\) = equivalent concrete thickness;
\(L\) = span of beam;
\(E\) = modulus of elasticity of floor system;
\(I\) = full uncracked moment of inertia of transformed section.

Clifton (1989) recommends Murray's method with modifications for New Zealand usage. An alternative expression for determining \(N_{eff}\) derived by Murray was included. Recommendations were also provided on how to achieve composite action of the primary girders with the concrete topping.

Tolaymat (1988) developed a computer program to calculate the response of a floor system excited by any time-dependent force. The program can study the response of exact heel drop impact, consecutive heel drops applied at equal intervals or the response under exciting forces other than heel drop. The program was used to analyse the behaviour of the data acquired by Murray (1981) and the following conclusions were made:
(a) a floor system will be acceptable if there is adequate damping to reduce the initial impact by 85% before the second heel drop is applied;

(b) a system with insufficient damping to reduce the initial impact amplitude by 85% before the second heel drop is applied, but $A_o f_o < 0.05$, where $A_o$ is in inches will be within the slightly perceptible range and hence the floor may still be acceptable;

(c) If $A_o f_o$ in (b) is $> 0.05$, the floor will be unacceptable.

Wyatt (1989) presented two methods for assessing the vibration behaviour of floors in steel framed buildings subjected to walking excitation. The response to the harmonic resonant component of the walking forces is to be assessed for floors of fundamental natural frequency less than 7 Hz. This method is similar to Allen's (1990). Wyatt stated that the floor beam support conditions and continuity may directly affect the effective modal mass by bringing into play more than one floor beam span. The response factor,

$$R = \frac{680000C}{MBD L_{eff}} \quad ........... [14]$$

where $M$ = floor mass in kg/m$^2$;
$D$ = the damping as % of the critical;
$C$ = the fourier component factor
$= 0.4$ if $f_o$ is between 3 and 4 Hz;
$= 1.4 - 0.25f_o$ if $f_o$ is between 4 and 4.8 Hz;
$= 0.2$ if $f_o$ is greater than 4.8 Hz;

$B$ and $L_{eff}$ are the effective floor width and span in m respectively and the way of estimating them is given by Wyatt.

For floors with a fundamental natural frequency greater than 7 Hz, the response to the heel impact component of the walking pace is to be assessed. This method is similar to that recommended in Appendix G CSA, 1984). The response factor,

$$R = \frac{30000}{MBL} \quad ........... [15]$$

where $B$ = the lesser of the secondary floor beam spacing or 40 $t_c$ in m;
$L$ = floor span in m. For continuous construction, $L$ is the larger of the span under consideration or of an adjacent span.

The limiting response factor, $R$ has been derived in conjunction with BS 6472 (BSI, 1984). $R$ should not exceed 4, 8 and 12 for a special office, general office and busy office respectively.

Allen's (1976, 1977) method can be used for all types of floors while both the Murray's and Wyatt's method were developed specifically for composite steel-concrete floors. Murray (1990) has however applied his method to insitu or precast concrete floors and found that the method can also be used for evaluating the response of such floors. Pending further experimental investigation to be undertaken by the author to determine which method is the most appropriate for New Zealand floors, these methods are all recommended for use in design.
3.0 REMEDIAL MEASURES

Most of the remedial measures discussed here can be used during the design stage, and should be considered when there is indication of a possible vibration problem.

Remedial measures for floor vibration problems generally can be applied either to the source, the transmission path, or the receiver. At the source, the aim is to isolate or reduce the excitations that are being generated from the floor system, for example, relocating the excitation elsewhere. In a floor with multiple activities, Clifton (1989) recommends isolating the area subjected to continuous excitations from the remainder of the structure using isolated columns. Separating this floor from other floors and installation of sound insulation would avoid vibration and sound propagation.

The transmission path can reduce vibrations by introducing barriers or discontinuities to the waves that reach a floor. The propagation of vibration waves, such as from walking can occur both longitudinally and transversely, but are generally more predominant in the latter direction. The propagation of vibration waves can be disrupted by:

(a) introducing unequal beam/floor spans;
(b) using unequal spans at alternate bays;
(c) installation of a non-loadbearing prop at midspan of floor.

Providing simple supports to the floor will avoid wave propagation beyond the excited floor span, but vibrations will still be felt within the span.

If the frequency of excitation, \( f \) equals the natural frequencies of the floor \( f_n \), then resonance occurs. When \( f/f_n \) approaches unity, the problems resulting from vibrations will be increased. Therefore, the higher the natural frequencies of floor, particularly the fundamental natural frequency \( f_0 \), the better is its performance when subjected to dynamic vibration loads. For any Single Degree of Freedom floor system, the fundamental natural frequency,

\[
f_0 = \frac{1}{2\pi} \sqrt{\frac{K}{M}}
\]

............ [16]

where \( K \) = the floor stiffness;
\( M \) = the mass of the floor plus a percentage of the live load.

Under a concentrated load, \( K' \) is proportional to \( EI/l^3 \) where \( E \) is the modulus of elasticity; \( l \) is the span, and \( I \) is the second moment of inertia. From this, it may be seen that increasing the stiffness of the floor reduces vibration problems by increasing the fundamental frequency. The floor stiffness can be increased by increasing the \( E \) or \( I \) values or decreasing the floor span.

The general measures to control floor vibration problems are therefore:

(a) isolating the excitation source from the floor system;
(b) modifying the boundary conditions to minimise the propagation of vibration waves;

(c) increasing the fundamental natural frequency of the floor system to avoid resonance under continuous vibrations by stiffening of the floor (increasing K by increasing I; decreasing the span of the floor; or reducing the mass);

(d) increasing the damping of the floor system under transient vibration;

(e) increasing mass to reduce the response to impulsive loads (Rainer, 1984)

3.1 Light Floors

For light-weight floors, Allen et al (1987) recommends providing sufficient stiffness against footstep deflection to reduce vibration problems. Ohlsson (1984) claimed that although typical displacement amplitudes due to a person walking on an ordinary timber floor are of the order of 0.5-2 mm, a very small movement at a beam support reduces the effective static stiffness substantially and restraints at the support should be considered as a means of control.

Ohlsson (1984) claimed that using herring-bone strutting showed only a small increase in transverse stiffness. This is because of the shrinkage of the timber in combination with alternations of loading directions. In addition, normal forces which are supposed to be transmitted to adjacent joists by contact did not materialise, since contact areas for transmission of forces are small. Solid blocking between joists is more effective than herring-bone strutting but their effectiveness is highly dependent on the degree of fixity that can be achieved (Onysko and Jessome, 1973).

Ohlsson (1982) reported on tests on a transverse stiffener installed at midspan and concluded that the method doubled the effective stiffness of the floor system under a concentrated force. The transverse stiffener was made up of an I-beam (hardboard or plywood web) which was 25-50 mm deeper than the main joists and was recessed to accommodate them. Adhesive was applied to the top of the transverse beam and it was then inserted between the joists.
3.2 Heavy Floors

Allen and Swallow (1975) claimed that contrary to popular belief, stiffening these floors is largely ineffective in "Transient Vibration Control" as this does not appreciably change the annoyance potential.

Allen et al's (1977) study of concrete floors stated that the contribution of non-structural elements appeared to be less effective in increasing the damping ratio than in steel joist metal deck with an insitu concrete topping.

Adding artificial structural dampers which usually includes the use of a viscoelastic material, during or after construction is another method to control undesirable vibrations in floors. However, only a few successful applications in actual buildings have been reported.

The installation of a non-loadbearing prop at midspan of the floor can also reduce the annoyance potential from vibrations. Such props have $\frac{1}{r_y}$ ratios of greater than 300, where $r_y$ is the radius of gyration of the minor axis, which would buckle laterally when loaded. This method is only possible where the occupants can cope with the additional prop.

To prevent resonance from continuous vibrations, increasing the stiffness of the structural system and reducing the mass (increases the fundamental natural frequency) are possible solutions. Another solution is to add vibration absorbers tuned to the same frequency as the natural frequency of the floor (Bachmann and Ammann, 1987). A vibration absorber is an oscillator of much smaller mass ($m$) than that of the floor structure ($M$), but with the same natural frequency. The optimum damping ratio for the absorber should be of the same order of magnitude as the effective damping ratio of the floor structure which is equal to $0.5\sqrt{\frac{m}{M}}$ (Allen and Pernica, 1984). Damping control becomes ineffective as the damping ratio moves away from this optimum. It should be noted that while this method has been demonstrated in the laboratory, successful application in buildings has not been reported.

4.0 RECOMMENDATIONS

4.1 Design Methods

Three criteria of design were encountered during the survey namely, resonance, stiffness and impulsive response. Resonance from crowd loading occurs in floors with uniform rhythmic activities where the fundamental natural frequency of the floor is less than about 8 Hz. Harmonic resonant response can also result from walking vibrations for floors below 8 Hz.

For floors with their fundamental frequency greater than 8 Hz, impulsive response governs the design. A static stiffness criterion may be sufficient for floors with frequencies greater than 20 Hz.

4.1.1 Continuous vibrations

For continuous vibration resulting from crowd loadings, Allen et al's (1985) procedure is recommended.
4.1.2 Transient vibrations

Chui and Smith's (1987) method is recommended for the design against transient vibrations in light floors. The requirement that floors have means of enhancing stiffness in the across-joists directions needs to be fulfilled.

The heel impact criterion (CSA, 1984), Wyatt (1989) and Murray's (1989) methods are recommended for the design against transient vibrations in heavy floors. The scope of each method must be noted before application. The CSA method applies to floors with spans greater than 7 m (frequency less than 10 Hz), while Murray's method is valid for floors with damping between 4-6% critical.

4.2 Remedial Measures

4.2.1 Light floors

The suggested remedial measure is to increase the transverse stiffness. The transverse stiffness of floors can be increased by the following methods:

(a) solid blocking between joists and support of edge joists;
(b) increasing flexural stiffness of the flooring by laying the stronger axis of the flooring perpendicular to the direction of joists;
(c) adding a loadbearing layer on the underside of joists (I-beam rather than T-beam effect) (Ohlsson, 1984);
(d) adding a ceiling, which also increases the effective damping of floors (Onysko and Jessome, 1973; Ohlsson, 1982).

4.2.2 Heavy floors

The suggested remedial measures for transient vibrations are: increasing the damping by adding partitions, or increasing the mass.

For continuous vibrations, the recommended measure is to increase the fundamental natural frequency of the floor (increasing its stiffness or decreasing its mass).

5.0 SUMMARY

A literature survey has been carried out to identify the available design methods, acceptability limits and remedial measures for occupant-induced vibrations in floors. A brief description of each method was given and discussed. The types of occupant-induced vibrations can be classified as either continuous (steady-state) or transient (impulsive). The floors were classified as either light or heavy-weight floors.

Design methods were recommended for continuous and transient vibrations for both light and heavy floors. Remedial measures suggested involved increasing transverse stiffness for light-weight floors, and either
increasing damping for transient vibrations, or increasing the fundamental frequency of the floor for continuous vibrations of heavy floors.

Some of the reasons why the floor damping recommended overseas should be applied with caution in New Zealand have been outlined. The next phase of this project is to carry out field testing of New Zealand floors to quantify these values. Calculated and measured natural frequencies of the floors will be compared. The different recommended design methods will be compared to ascertain which yield the best results.

PRIMARY REFERENCES


REFERENCES


Murray, T.M. 1990. Personal communication.


APPENDIX A: Equations for the Fundamental Natural Frequency

A.1 Isotropic Floors (simply supported on all four edges)

\[ f_o = \frac{\pi (L^2 + B^2)}{2L^2 B^2} \sqrt{\frac{D^*}{M}} \quad \ldots \ldots [17] \]

\[ D^* = \frac{E t^3}{12 (1-\mu^2)} \quad \ldots \ldots [18] \]

where \( B \) and \( L \) are the width and span of the floor respectively;
\( D^* \) = flexural rigidity of the floor;
\( E \) = modulus of elasticity
\( M \) = mass per unit area of the floor;
\( \mu \) = poisson's ratio;
\( t \) = thickness of floor.

A.2 Heavy Floors (one-way action)

\[ f_o = K' \sqrt{\frac{gEI}{WL^3}} \quad \ldots \ldots [19] \]

where \( E \) = modulus of elasticity, MPa;
\( g = 9810 \text{ mm/s}^2; \)
\( L \) = span in mm;
\( I \) = is the full uncracked moment of inertia of the transformed section; assuming a concrete flange of width equal to beam spacing refer Figure 6, mm^4;
\( W \) = dead weight plus 10-25\% live load of T-section, N;
\( K' = 1.57 \) for simply supported beams or for continuous beams with reasonably regular spacing
\( = 0.56 \) for cantilevered beams
\( = \) as given in Figure 7 for overhanging beams
\( = \) as given in Figure 8 for continuous beams

The \( I \) value should be calculated based on full composite action even if the slab is not structurally connected, and rests on the supporting member. This is logical since the magnitude of the forces causing vibrations is not sufficient to overcome the friction force between the elements. Similarly, the full uncracked moment of inertia should be used because the stresses resulting from vibration forces would be too small to cause cracking of the supporting concrete members.

Where the one-way system is supported on steel girders (two-way floor system), the floor frequency will be smaller than that calculated for the one-way system. The frequency of the new system can be approximated by:

\[ \frac{1}{f_s^2} = \frac{1}{f_o^2} + \frac{1}{f_p^2} \quad \ldots \ldots [20] \]
where $f_s$ = fundamental natural frequency of the two-way floor system;
$f_o$ = fundamental natural frequency of the one-way system from equation [19];
$f_p$ = fundamental natural frequency of primary members.

A.3 Light Timber Floors (simply supported)

Chui (1986) used the Rayleigh method to derive the fundamental natural frequency of a floor, ($f_o$):

$$f_o = \frac{\pi \sqrt{E_j I_j (n-1)}}{2L^2 \sqrt{f_s t B + \ell_j A (n-1)}} \quad \ldots \ldots \quad [21]$$

where $E_j$ = joist modules of elasticity;
$I_j$ = second moment of area per joist;
$n$ = number of joists;
$t$ = sheathing thickness;
$A$ = cross sectional area per joist;
$s$ = sheathing density;
$\ell_j$ = joist density;
$B$ = width of the floor;
$L$ = span of floor.

A.4 Beams (simply supported, fixed, cantilever)

The fundamental frequencies of beam (neglecting shear) is shown in Table 6 taken from Steffens (1974).
Figure 1a  Some Light-weight Floors in New Zealand

Timber joists  Twinaplate joists - timber flanges with corrugated steel web

Precast ribs with timber infill  Precast Double Tees

Precast Dycore  Insitu Slab

Precast Flat Slab

Figure 1b  Typical Heavy Floors in New Zealand
Figure 2  Acceleration Curves: Foot-to-head direction (from Cooney and King (1988))
Figure 3  Annoyance Criteria for Floor Vibrations (Residential, School, Office, Occupancies), from CSA (1984)
LEGEND:

+ = motion not noticed
○ = motion noticed but floor acceptable
◇ = motion noticed and opinion undecided
◆ = motion noticed and floor unacceptable.

Figure 4  Plot of serviceability criterion with maximum deflections for 98 floors tested in the field after Onysko (1988)
Figure 5  Preliminary proposal for classification of the response of a floor construction to an impact load by Ohlsson (1984)
Figure 6  Tee-Beam Model for Computing Transformed Moment of Inertia
(from Murray (1989))

Figure 7  Frequency Coefficients for Overhanging Beams
(from Murray (1989))
Figure 8 Frequency Factor $K'$ for Continuous Beams after Wyatt (1989)
<table>
<thead>
<tr>
<th>Activity and Construction</th>
<th>Forcing Frequency $f$, Hz</th>
<th>Weight of Participants $w_p$, kPa</th>
<th>Dynamic Load $q w_p$, kPa</th>
<th>Total Weight $w_t$, kPa</th>
<th>Minimum Structural Frequency $f_s$, Hz Equation (4)</th>
<th>Maximum Static Deflection $\Delta$, under $aw_p$, mm Equation (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dancing and Dining $a/g = 0.02$</td>
<td>3</td>
<td>0.6</td>
<td>0.3</td>
<td>5.6</td>
<td>6.4</td>
<td>0.43</td>
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<td>Solid concrete 5 kPa</td>
<td>3</td>
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<td>0.3</td>
<td>3.1</td>
<td>8.1</td>
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<td>Steel joist 2.5 kPa</td>
<td>3</td>
<td>0.6</td>
<td>0.3</td>
<td>1.3</td>
<td>12.0</td>
<td>0.52</td>
</tr>
<tr>
<td>Wood 0.7 kPa</td>
<td>3</td>
<td>0.6</td>
<td>0.3</td>
<td>0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Lively Concert or Sports Events $a/g = 0.05$</td>
<td>3</td>
<td>1.5</td>
<td>0.4</td>
<td>6.5</td>
<td>4.8</td>
<td>0.85</td>
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<tr>
<td>Solid concrete 5 kPa</td>
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<td>1.5</td>
<td>0.4</td>
<td>4.0</td>
<td>5.7</td>
<td>1.00</td>
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<tr>
<td>Steel joist 2.5 kPa</td>
<td>3</td>
<td>1.5</td>
<td>0.4</td>
<td>2.2</td>
<td>7.2</td>
<td>1.14</td>
</tr>
<tr>
<td>Wood 0.7 kPa</td>
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<td>1.5</td>
<td>0.4</td>
<td>0.7</td>
<td>0.7</td>
<td></td>
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<tr>
<td>Jumping Exercises $a/g = 0.05^{(1)}$</td>
<td>6</td>
<td>0.4</td>
<td>0.1</td>
<td>5.4</td>
<td>7.3</td>
<td>0.11^{(2)}</td>
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<tr>
<td>Solid concrete 5 kPa</td>
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<td>2.9</td>
<td>8.3</td>
<td>0.16^{(2)}</td>
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<tr>
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<td>1.1</td>
<td>11.7</td>
<td>1.29</td>
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<tr>
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<td>0.4</td>
<td>0.1</td>
<td>0.7</td>
<td>0.7</td>
<td></td>
</tr>
</tbody>
</table>

Notes to Table

(1) The criteria are applied to 2 cases (i.e. $f = 3$ Hz and $f = 6$ Hz) and the governing case is used.

(2) Corresponding deflections under 0.6 kPa are 0.66 mm for solid concrete and 0.96 mm for steel joist.

Table 1 Application of vibration criteria to different activities and floor construction (from Allen et al (1985))
<table>
<thead>
<tr>
<th>Occupancy/activity</th>
<th>Frequent steady-state RMS acceleration</th>
<th>In frequent Transient Damping</th>
<th>Peak acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hospitals</td>
<td>0.002</td>
<td>3</td>
<td>0.005</td>
</tr>
<tr>
<td>Laboratories</td>
<td>0.005</td>
<td>6</td>
<td>0.01</td>
</tr>
<tr>
<td>Hotels and multi-family apartments</td>
<td></td>
<td>12</td>
<td>0.02</td>
</tr>
<tr>
<td>Offices</td>
<td>0.01</td>
<td>3</td>
<td>0.05</td>
</tr>
<tr>
<td>School rooms</td>
<td>12</td>
<td>6</td>
<td>0.10</td>
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<tr>
<td>Libraries</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dining rooms and restaurants</td>
<td>0.02</td>
<td>12</td>
<td>0.20</td>
</tr>
<tr>
<td>Assembly areas, theaters</td>
<td></td>
<td>3</td>
<td>0.05</td>
</tr>
<tr>
<td>One- and two-family dwellings</td>
<td>12</td>
<td>6</td>
<td>0.10</td>
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<tr>
<td>Stores and shopping centers</td>
<td>0.02</td>
<td>3</td>
<td>0.05</td>
</tr>
<tr>
<td>Manufacturing, warehouses</td>
<td></td>
<td>6</td>
<td>0.10</td>
</tr>
<tr>
<td>Walkways, stairs and exitways</td>
<td></td>
<td>12</td>
<td>0.20</td>
</tr>
<tr>
<td>Dance halls and ballrooms</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recreational areas</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Gymnasiums</td>
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<td></td>
<td>N.A.</td>
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<tr>
<td>Stadiums and arena bleachers</td>
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<td>Special assembly structures</td>
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Table 2  Suggested Acceleration Limits by Occupancy from Ellingwood and Tallin (1984)
(Refer section 2.3)

<table>
<thead>
<tr>
<th>Harmonic</th>
<th>Office Floors</th>
<th>Shopping Malls</th>
<th>Footbridges</th>
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<tr>
<td>First</td>
<td>28</td>
<td>9.3</td>
<td>4</td>
</tr>
<tr>
<td>Second</td>
<td>14</td>
<td>4.7</td>
<td>2</td>
</tr>
<tr>
<td>Third</td>
<td>7</td>
<td>2.3</td>
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<td>Fourth</td>
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Table 3a  Minimum Values of $DW$ (kN), In [2b] for Satisfactory Performance from Allen (1990)

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<th>D</th>
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<td>Footbridge</td>
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</tr>
<tr>
<td>Bare Floor</td>
<td>0.015</td>
</tr>
<tr>
<td>Finished floor with ceiling, ducts, flooring and furniture</td>
<td>0.03</td>
</tr>
<tr>
<td>Finished floor with partitions</td>
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Table 3b  Recommended Damping Ratios, $D$, for Eqn [2b] from Allen (1990)
<table>
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<th>$f'$ (Hz)</th>
<th>X</th>
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</thead>
<tbody>
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<td>8</td>
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<tr>
<td>9</td>
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<tr>
<td>10</td>
<td>44.0</td>
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<tr>
<td>11</td>
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<tr>
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<td>45.6</td>
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<tr>
<td>13</td>
<td>44.5</td>
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<tr>
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<tr>
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Table 4  X values  
from Chui and Smith (1988)
### Table 5: Dynamic Factors for Heel-Drop Impact after Murray (1989)

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<th>$\omega_0$, Hz</th>
<th>$(DLF)_{max}$</th>
<th>$\omega_0$, Hz</th>
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</table>
**Table 6**  Fundamental frequencies of beams (neglecting shear)
after Steffens (1974) (Refer A5)

<table>
<thead>
<tr>
<th>Case</th>
<th>Beam type</th>
<th>Loadings</th>
<th>Static deflection (general expression)</th>
<th>Value of constant (C)</th>
<th>Correction to be added to load P to allow for weight (Wo) of beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cantilever</td>
<td>At end</td>
<td>$PL^3/3EI$</td>
<td>15.8</td>
<td>$0.225 , Wo$</td>
</tr>
<tr>
<td>2</td>
<td>Cantilever</td>
<td>UDL</td>
<td>$PL^3/8EI$</td>
<td>19.6</td>
<td>$Wo$</td>
</tr>
<tr>
<td>3</td>
<td>Simply-supported</td>
<td>Central</td>
<td>$PL^3/48EI$</td>
<td>15.8</td>
<td>$0.50 , Wo$</td>
</tr>
<tr>
<td>4</td>
<td>Simply-supported</td>
<td>UDL</td>
<td>$PL^3/76.8EI$</td>
<td>17.7</td>
<td>$Wo$</td>
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<td>5</td>
<td>Built-in(encastre)</td>
<td>Central</td>
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<td>$0.375 , Wo$</td>
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<td>UDL</td>
<td>$PL^3/384EI$</td>
<td>17.7</td>
<td>$Wo$</td>
</tr>
</tbody>
</table>

**Notes**

- P is the point load or total distributed load
- L is the span
- E is the modulus of elasticity of the material
- I is the second moment of area of beam section
- Natural frequency (Hz) = $C/\Delta_s$, where static deflection ($\Delta_s$) is in mm

(Consistent units for P, L, E and I must be used to determine $\Delta_s$)
A literature survey of design methods for floors
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HEAD OFFICE AND RESEARCH CENTRE

Moonshine Road, Judgeford
Postal Address - Private Bag, Porirua
Telephone - (04) 357-600, FAX 356-070

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AUCKLAND
Telephone - (09) 5247-018
FAX - (09) 5247-069
290 Great South Road
PO Box 17-214
Greenlane

WELLINGTON
Telephone - (04) 357-600
FAX 356-070
Moonshine Road, Judgeford

CHRISTCHURCH
Telephone - (03) 663-435
FAX - (03) 668-552
GRE Building
79-83 Hereford Street
PO Box 496

DUNEDIN
Telephone - (03) 4740-454
FAX - (03) 4778-436
6 Hanover Street
PO Box 1635

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