DYNAMIC RESPONSE OF STRUCTURAL TIMBER FLOORING SYSTEMS

by

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Abstract

The dynamic response of structural timber flooring systems can cause vibrational serviceability problems in terms of discomfort experienced by the occupants. A unified method to control timber floor vibrations has not been established to-date. The vibration problem is manifold. The complexity and the limited amount of research with respect to timber floor vibrations have shown an urgent need for further investigations.

This thesis has focused on the effects of structural and non-structural modifications on the dynamic performance of timber flooring systems by using experimental data from sixty-seven full-scale flooring systems for analytical investigations so as to identify structural configurations and vibration parameters, which are promising to further the design against disturbing vibrations. The collected data have also been used to identify weaknesses of current design criteria and to build and validate a finite element (FE) model for eigenproblem analyses of timber I-joist floors. The experimental work has been carried out with support from industry, and part of the investigations with respect to the design criteria has been conducted as Visiting Scientist within a Short Term Scientific Mission of COST Action E55 at VTT - Technical Research Centre of Finland in Espoo, Finland.

The significant effects of floor make-up and different configurations on their dynamic response are examined, with specific interest to stiffen dynamically sensitive locations targeted, and the most promising designs (configurations) are identified. The important effects of damping on the dynamic performance of flooring systems are addressed by determination of damping ratios from the full-scale experimental work. The results were then used to perform a series of statistical studies to identify and recommend more appropriate damping ratios for design of bare light-weight timber flooring structures based on a number of distinct structural properties. The computer-based finite element analysis has been successfully used to model a series of timber flooring systems incorporating timber I-joists for predicting modal parameters and their relative changes due to structural modifications. The analysis has demonstrated the significant influence of assigning spring stiffness at the supports and at the interface of deck and joists on the floor responses.

Overall, this research has helped to achieve a much broader knowledge and greater understanding of dynamic response and vibrational characteristics of timber flooring systems, and has made a contribution to identifying improved structural design and

furthering vibration prediction and assessment. Undertaking of any such measures and future work as suggested in this thesis could significantly contribute to the improvement of the structural design and the design to Eurocode 5 if results are incorporated in future revisions. This would lead to fewer nuisances for residential occupants and enhanced quality of life.

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Publications

During the course of this PhD research programme the following papers were published:

WECKENDORF J., ZHANG B., KERMANI A. and REID D. 2005. Dynamic response of timber floors. *Proceedings of the 2nd PRoBE Conference.* Glasgow, UK.

WECKENDORF J. 2006. Evaluation of the Dynamic Response of UK Timber Floors. *Synopses of IStructE Young Researchers' Conference.* London, UK.

WECKENDORF J., ZHANG B., KERMANI A., DODYK R. and REID D. 2006. Assessment of vibrational performance of timber floors. *Proceedings of the 9th World Conference on Timber Engineering.* Portland, Oregon, USA.

WECKENDORF J. 2007. *A Comparison of Design, Construction and Dynamic Performance of Timber Floors in the UK and Finland. Short Term Scientific Mission of COST Action E55: Modelling the Performance of Timber Structures.* COST ACTION E55 (Ed).

WECKENDORF J. 2007. Investigations of dynamic behaviour of timber floors. *Proceedings of the FECCI Research Conference.* Edinburgh, UK.

WECKENDORF J., ZHANG B., KERMANI A. and REID D. 2007. *Research on the Vibrational Performance of Timber Flooring Systems at Napier University in the UK. Report for 2nd Workshop of COST Action E55 - Modelling of the Performance of Timber Structures.* Eindhoven, The Netherlands.

WECKENDORF J., ZHANG B., KERMANI A. and REID D. 2007. Vibrational behaviour of timber floors – experimental investigations. *Proceedings of the 3rd PRoBE Conference.* Glasgow, UK.

WECKENDORF J., ZHANG B., KERMANI A. and REID D. 2008. Effects of local stiffening on the dynamic performance of timber floors. *Proceedings of the 10th World Conference on Timber Engineering.* Miyazaki, Japan.

WECKENDORF J., ZHANG B., KERMANI A., REID D. and ANDERSEN P. 2008. Damping characteristics of timber flooring systems with respect to low-frequency vibration modes. *Proceedings of the 10th World Conference on Timber Engineering.* Miyazaki, Japan.

1. Introduction

The use of timber as a structural material for residential buildings has a long tradition. Wood was considered an adequate building material already millenniums ago. Ever since, a steady development process has taken place in the production and construction practices with the movement from using solid timber elements to engineered wood products, from traditional timber joints such as mortise and tenon or dovetail notches to joints with metal fasteners and from small-scale buildings to large-span structures. The traditional building practices have evolved and in parts have been replaced by methods of modern construction. Only in the United Kingdom (UK), currently there are already 200,000 new homes built per year, of which 75% incorporate timber floors, nearly 80% being constructed with timber I-joists, more than 15% with metal-web joists and a minority with solid timber joists. All these structures are usually built to a satisfactory degree with respect to stability and load bearing capacity.

However, the modernisation of the products and structural systems and the more economic construction practices with more flexible and lighter-weight structural elements have amplified serviceability issues for the structures. In particular, timber flooring systems in buildings can suffer from inadequate vibrational performance when excited, e.g. by human walking, which in most cases is rather harmless to the structural stability but can cause discomfort to the occupants.

This serviceability aspect has been gaining sparse attention and the modernisation of the serviceability design criteria has been progressing less rapidly than the modernisation of structural systems. Existing design criteria have been found to be inadequate to control floor vibrations to an acceptable degree.

The issue of unsatisfactory vibrational performances of flooring systems has given rise to several research activities worldwide, resulting in a number of design guides and standards by making use of a number of parameters to control the vibrational performance of flooring systems. A unified method to be accepted worldwide has not been established to date. The probably most recognised and influential research in the field of timber floor vibrations carried out in Europe dates back to the 1980s (Ohlsson 1982, 1988; Chui 1987).

In Europe, in recent years, Eurocodes (ECs) have been established to serve as pan-European standards in the form of harmonised design criteria within the member countries to build a common basis for design, research and development. The design of timber structures is covered in Eurocode 5 (EC5). The criteria associated with timber floor vibrations are part of the serviceability limit states (SLS) of this standard and are based on the above-mentioned research carried out by Ohlsson (1982, 1988).

These adopted criteria are not fully indisputable, which has already led to modifications of the EC5 design rules in the National Annexes (NAs) to EC5 in some member countries, which is further specified in the literature review (Chapter 3). The design rules of the ECs and other standards and guides can aid to limit the number of floors with unacceptable vibrational performance but cannot fully avoid them.

This is not only a result of the limited research carried out in this field but also of the complexity of this subject for accurately predicting the dynamic parameters and carefully selecting suitable parameters for the design, while simplifying complex formulae towards user-friendly equations. After all, the final identification of floors as being vibrating satisfactorily or unsatisfactorily depends on the subjective ratings of occupants, who will be exposed to the vibrations. It will therefore be hard to establish design criteria, which can fully separate acceptable floors from unacceptable ones. However, with the ongoing modernisation in construction there will be an ongoing challenge to advance the structural design for minimising the amount of flooring systems that will be rated as being vibrating unsatisfactorily.

The lack of in-depth understanding of dynamic response of flooring systems and the effects of flooring components individually and collectively, the lean design techniques and related side effects of modern methods of construction continue to cause increased levels of concern regarding serviceability limit states and as such raises the urgent need for in-depth investigations in the field of timber floor vibrations.

1.1 Overview of floor vibration issues and scope of study

1.1.1 General issues of floor vibration research

To set a structure into vibration, it can be excited e.g. by wind loads, rotating machineries or human activities such as walking, running, dancing or jumping. The most common types of excitation with regard to timber floors are human walking and rotating machineries such as washing machines. The issue of vibrations induced by machineries is usually solved by isolating the machinery from the floor. However, the possibility of exciting the floors through human activities remains large since walking excitation cannot easily be isolated from the structure. It then needs to be considered whether forcing frequencies coincide with or are close to the natural frequencies of the system. In such cases, the forced vibration amplitudes can be significantly amplified (see also Chapter 2). Vibration amplitudes are expressed by dynamic displacement, velocity or acceleration. The parameters describing the dynamic characteristics of a structure are the modal parameters: modal frequencies, modal damping and modal shapes.

Occupants of dwellings often are not only the cause of the vibrations. It is their sensitivity to floor vibrations, which results in the serviceability issues relating to discomfort. However, the occupants' rating of vibrations as acceptable or not is highly subjective and depends on a number of (partially uncontrollable) aspects: the position of a person, the type of activity, the time of the day, the location, etc.

Different design guides introduce different parameters for controlling floor vibrations. Not all criteria account for all properties that influence the dynamic response of a system: mass, stiffness and damping. Simplification of complex dynamic equations without losing an acceptable degree of accuracy is hard to achieve.

1.1.2 Research focus

Due to the complexity of this subject, this research has focused on a number of key aspects such as the effect of different structural and non-structural modifications of modern timber flooring systems constructed with composite timber I-joists or open-web joists on their dynamic response, with main interest in the modal parameters and static deflection. Extensive experimental studies have been carried out to determine these parameters for a detailed analysis. The predictability of response parameters has been examined with respect to design criteria and finite element method.

Light-weight timber flooring structures usually possess natural frequencies outside the most critical range of excitation frequencies from walking. Thus, the vibration response is not necessarily dominated by distinct modes with the highest vibration amplitude magnification. The overall influential vibration response is usually composed of several frequency components. Since the mode shapes and thus the deflection patterns may vary for different natural frequencies, the vibration sensed can also vary with the location of the human on the floor. Therefore, the impact of structural and non-structural components on the individual modes of vibration within the frequency range of interest is one of the key aspects in this work. Of large interest therein is one particular practical solution to tackle vibration problems, which are related to certain local floor areas.

Damping is a very important factor in vibration. Its significance has been detailed in Chapters 2, 3 and 6 to 8. Damping controls the amplitude magnification when the excitation frequency and the natural frequency of the system are close or coinciding. Damping also determines how quickly the amplitude of free vibration attenuates. Past research reports have identified a higher tolerance of humans with respect to initial vibration amplitudes if it is damped within short duration (Lenzen 1966; Ohlsson 1988). Damping is sometimes used in design against disturbing vibrations. In the BS EN 1995-1-1:2004 (EC5-1-1), one specific damping ratio is provided and used for controlling unit impulse velocity responses of all types of rectangular timber flooring systems simply supported along all four edges with natural frequencies above 8 Hz; and as such indicates its importance as a very influential design parameter. Damping, however, can vary, depending on structural details and loading conditions. Hence, the modal parameter damping has been of great interest. The information obtained on damping will be analysed so as to identify suitable values for design, whose selection can be dependent on a few distinct construction parameters.

The vibration response of timber floors is not easily predicted with a high degree of accuracy using simplified design equations. Even complex mathematical methods may not always yield excellent results. The predictability of vibration parameters and potential effects on the vibration assessment have been examined. The computer-based finite element method (FEM) has been used to model timber I-joist flooring systems for the prediction of natural frequencies, modal shapes and static deflection. The design criteria of EC5-1-1 and National Annex to BS EN 1995-1-1 2004 (UK NA to EC5-1-1) have been analysed using the results obtained from the tests on the open-web joist floors. Furthermore, investigations on the design criteria including the EC5-1-1 proposals and the guides in the NAs of the two European countries UK and Finland have been carried out during a Short Term Scientific Mission (STSM) of COST Action E55 at VTT - Technical Research Centre of Finland, in Espoo, Finland.

1.2 Objectives and methodology

This research has aimed to provide a better understanding of timber floor vibration serviceability with focus on state-of-the-art timber floor construction styles. The undertaken research has not only indicated potential structural improvements with respect to serviceability design but also has helped to illustrate the limitations of the currently available design criteria.

The final aim is to identify construction styles and design variables, which are promising to further enhancing structural design with corresponding serviceability criteria. Recommendations are made towards the design of new and existing structures and towards furthering the assessment of vibration. It is moreover aimed to provide a finite element model, which allows predicting the modal parameters of frequencies and shapes of timber I-joist floors reliably. The objectives can be summarised as follows:

- Filling gaps of information on the dynamic response of timber flooring structures in particular for those floors, which are constructed with engineered joists such as composite timber I-joists and metal-web joists, meeting current state-of-the-art construction techniques;

- Exposing weaknesses of current design criteria and proposing more suitable design parameters for the assessment of vibrational floor behaviour, focusing on the damping as a highly influential quantity;

- Furthering the enhancement of structural floor design by guiding through the benefits of a number of structural detailing techniques under consideration of different design aspects so as to select and follow suitable steps for the improvement of dynamic floor responses;

- Providing a finite element model for I-joist flooring systems to perform reliable eigenproblem analysis and demonstrating the importance of considering spring stiffness at the supports and at the interface of deck and joists in numerical modelling of composite timber flooring systems.

An operational modal analysis was carried out to determine the dynamic parameters of the flooring structures. The ARTeMIS Testor and Extractor were used for test conduction and data analysis respectively. Three identification techniques were available: the Frequency Domain Decomposition (FDD), the Enhanced FDD and the Stochastic Subspace Identification, performing the modal parameter estimation in the frequency domain, in the frequency and time domain and purely in the time domain respectively. These advanced and powerful techniques, since established for modal analysis, have been widely used in civil and mechanical engineering for the identification of modal parameters of structures, machineries, individual elements and bodies.

Before signal processing, the required data were collected by test conductions on a significant number of full-scale flooring systems. Therefore, the dynamic responses were measured at a suitable number of measurement points. The number of points was selected so as to obtain the mode shapes of all vibration modes of interest and appropriate averages of the quantities of modal frequencies and damping. Trials prior to the start of the experimental study showed that sweeping the floor surface with a brush was a useful method to yield broad-banded random excitation. It was also found that moving a trolley over the floor surface was an appropriate substitute of brush excitation. The static point load deflection was measured at floor centre and sometimes along mid-span of other floor joists using standard procedures. Steel sections were used to form the load whereas dial gauges or displacement transducers were employed to measure the deflections. More detailed information about the experimental approach and the data analysis can be found in Chapter 4.

The data analysis was extended to direct comparisons of the modal parameters for various structural modifications to evaluate the dimension of relative changes in response characteristics. Damping was furthermore considered in absolute terms to perform some statistical analyses for the identification of suitable damping ratios that can be used for the design, depending on a few distinct structural parameters.

To support the investigation of modal parameters and their relative changes due to structural modifications by numerical methods, the popular FEM software package LUSAS was utilised to model timber I-joist flooring systems. Special attention was focused on the effect of spring stiffness at the interface of different structural elements for their connection.

These investigations will help to further the design and assessment of structural timber flooring systems with respect to their dynamic responses by proposing appropriate structural design measures and more suitable damping ratios for the vibration assessment. They furthermore provide a suitable computer-based FE-model for numerical studies.

1.3 Thesis outline

To provide an introduction to system vibrations, some principles are summarised in Chapter 2, based on a single-degree-of-freedom system, which also serves as a basis for the modal analysis. Some general considerations of vibrational behaviour and expressions used in this thesis are also included. Relevant design guides and past research with respect to floor vibrations are reviewed in Chapter 3 with focus on the aspects of human sensitivity to vibrations, the effect of floor properties on the vibrational behaviour, the design against disturbing vibrations and the predictability of dynamic variables by the use of analytical and numerical models. Chapter 4 includes the experimental investigations. It describes the design of the various flooring structures tested, which have been categorised for different test series, and the methods for the static and dynamic tests. The available modal analysis techniques are explained with focus on the stochastic subspace identification. The results and the observed floor behaviours due to structural modifications are then detailed and discussed in Chapters 5 to 9 so that recommendations with respect to structural design against annoying floor vibrations could be given. Furthermore, the Chapters 6 and 7 include the investigation of current design criteria to identify their potential weaknesses. The results of statistical analysis on damping ratios to identify suitable values for design are shown in Chapter 8. Chapter 9 summarises and discusses the numerous results. It is followed by Chapter 10, presenting the construction of a computer-based finite element floor model and the subsequent prediction of modal frequencies, modal shapes and point load deflection and their correlations to measured results. The complete work is summarised in Chapter 11 followed by the conclusions in Chapter 12. The recommendations for future research, which are mainly based on findings from this project, are provided in Chapter 13.

2. Basic principles of vibrating systems

In this chapter principles about vibrating systems, especially single-degree-of-freedom (SDOF) systems, are described as a basis for understanding characteristics of dynamic structural behaviour.

2.1 Basics of simple harmonic motion

Firstly, there is the simple harmonic motion. A one dimensional system can exhibit simple harmonic oscillation after it is being displaced from its equilibrant force position. A restoring force occurs due to tension, compression or shear in the system or due to gravity. This restoring force makes the system moving back into the direction of its equilibrium position. As shown by Hooke's Law, the value of the force is proportional to the system's displacement in a linear-elastic system (Pain 1992):

$$F = -k\,y(t) \quad [\mathrm{N}] \tag{2.1}$$

where F is the force in [N], k is the stiffness in [N/m] and $y(t)$ is the displacement at time t in [m]. The restoring force causes acceleration, which brings the mass back to the equilibrium position. Acceleration is defined as the rate of change of velocity, normally measured in [m/s^2]. Newton's Second Law considers this effect as follows:

$$F = m\,\ddot{y}(t) \quad [\mathrm{N}] \tag{2.2}$$

where m is the mass in [kg] and $\ddot{y}(t)$ is the acceleration at time t in [m/s^2].

The system gathers momentum and continues to vibrate constantly if neglecting all energy losses. This kind of oscillation can be considered as free undamped oscillation. Therefore, the equation of motion (Eq. (2.3a)) is deduced from Hooke's Law and Newton's Second Law by rearrangement after letting Eq. (2.1) equalise Eq. (2.2), and is a linear homogeneous differential equation of second order:

$$m\,\ddot{y}(t) + k\,y(t) = 0 \tag{2.3a}$$

$$\ddot{y}(t) = -\frac{k}{m}\,y(t) \tag{2.3b}$$

$$\ddot{y}(t) = -\omega^2\,y(t) \tag{2.3c}$$

where m and k are constant. The property ω is the angular velocity or the natural frequency of the system which therefore is expressed as:

$$\omega = \sqrt{\frac{k}{m}} \quad \text{[rad/s]} \tag{2.4}$$

A solution to Eq. (2.3a) is:

$$y(t) = Y \sin(\omega t + \varphi) \tag{2.5a}$$

or

$$y(t) = Y \cos\varphi \sin\omega t + Y \sin\varphi \cos\omega t = A_1 \sin\omega t + A_2 \cos\omega t \tag{2.5b}$$

with

$$A_1 = Y\cos\varphi,\ A_2 = Y\sin\varphi,$$
$$\tan\varphi = \frac{A_2}{A_1},$$
$$Y = \sqrt{A_1^2 + A_2^2}$$

where the constants A_1, A_2 and φ are to be determined by the initial conditions. The constant Y is representing the amplitude and φ the phase angle in [rad]. If the general sine curve is considered, the maximum displacement is unity. The phase angle φ represents the initial position in the cycle of oscillation (Figure 2.1).

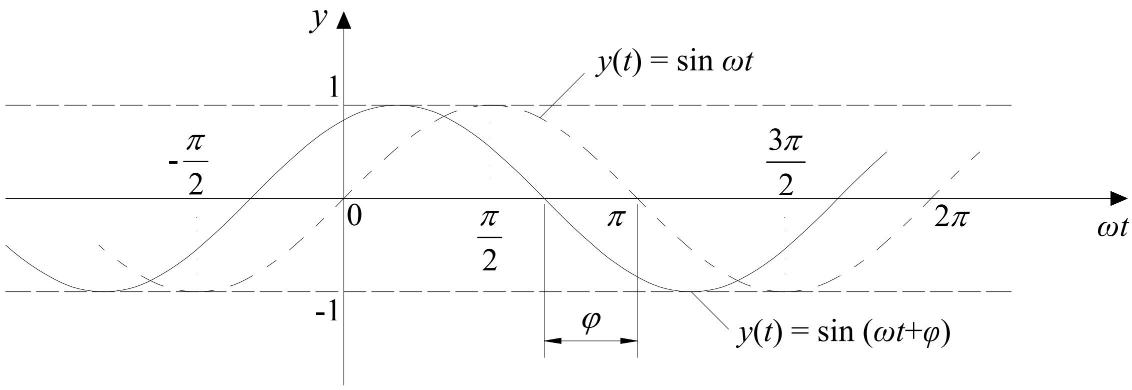


Figure 2.1: Phase shift of sine curve along the axis of abscissas (cf. Bartsch 1999)

Equation (2.4) shows that the frequency will be higher if stiffness increases, and if the mass increases, the resulting frequency will decrease. A frequency is often denoted in Hertz [Hz]. A frequency of 1 Hz is equivalent to 1 vibration (cycle) per second [1/s]. In this case the property is f. The relation between the properties ω and f is:

$$f = \frac{\omega}{2\pi} \quad \text{[Hz]} \tag{2.6}$$

The period of oscillation is expressed by T, which is the reciprocal of f (Bartsch 1999):

$$T = \frac{1}{f} = \frac{2\pi}{\omega} \quad \text{[s]} \tag{2.7}$$

Displacement in relation to the equilibrium position of a simply harmonic oscillating system, its velocity and acceleration at a certain time demonstrate the performance of the structure. The relationship between them is shown in Figure 2.2 where sine curves in three different graphs represent free vibration of a SDOF system in terms of the different expressions of motion.

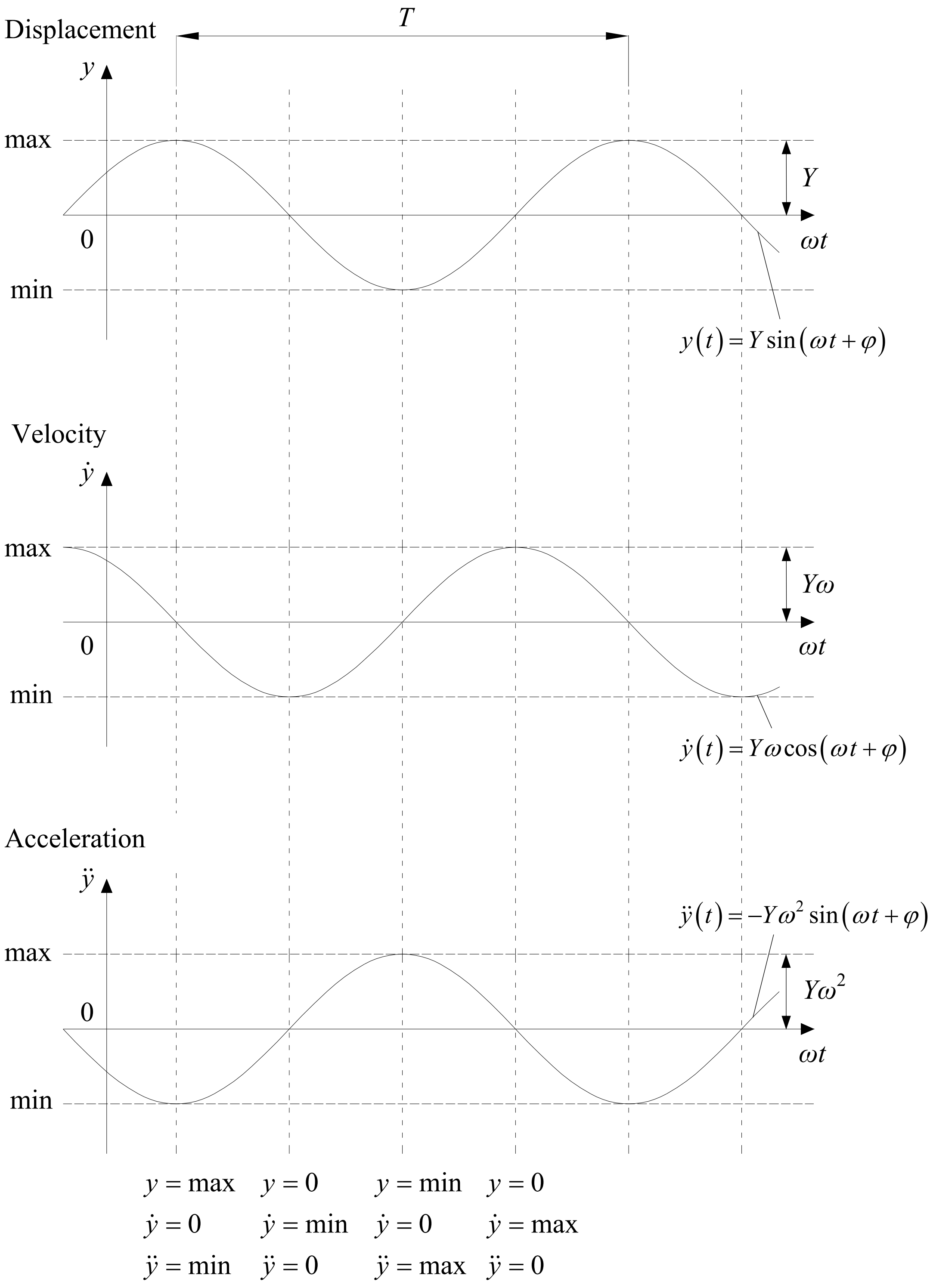


Figure 2.2: Oscillation of an undamped single-degree-of-freedom system: displacement, velocity and acceleration graphs (cf. e.g. Pain 1992)

The equilibrium position is the inflection point for the motion. The first derivation of the inflection point yields the minima and maxima whereas the second derivation is zero. Thus, as can be seen form Figure 2.2, the acceleration reaches its valley and velocity is zero if the amplitude of displacement reaches its peak. This in turn means that the velocity's curve reaches the valley and the acceleration is zero if the displacement reaches the equilibrium position. If the displacement is at its valley, then the velocity is zero again whereas the acceleration reaches its peak.

2.2 The properties and effects of damping

The descriptions given so far are related to linear free, undamped oscillations. An undamped vibration exists only in theory. It is, with the exception of initial amplitude, free from extraneous cause and continues to vibrate permanently. The amplitude into the deflected and into the opposite direction remains the same constantly. In reality energy is dissipated by friction, which avoids undamped vibrations. Damping is an effect that decelerates the movement of the system and often viscous damping is assumed, whereas viscous might be roughly outlined as a strong but finite resistance versus a motion. A viscous damping force is always acting in the opposite direction related to the motion and is proportional to the velocity (Pain 1992):

$$F = -c\,\dot{y}(t) \quad [\mathrm{N}] \tag{2.8}$$

where c is the damping coefficient in [kg/s] or [(Ns)/m].

Applying the damping force to Hook's Law and Newton's Second Law shows:

$$m\,\ddot{y}(t) = -k\,y(t) - c\,\dot{y}(t) \tag{2.9a}$$

or

$$m\,\ddot{y}(t) + c\,\dot{y}(t) + k\,y(t) = 0 \tag{2.9b}$$

where m, c and k are constant. This is the equation of motion for damped oscillation. A solution of the following form is assumed (Pain 1992; Thomson 1993):

$$y(t) = Ae^{\lambda t} \tag{2.10}$$

where A and λ are constants, and the number e is a mathematical constant, which is irrational and used for describing growth (or decay) behaviours. The solution can take three different forms, whereof each describes a different behaviour of $y(t)$ (Pain 1992), as will be shown subsequently.

Substitution of Eq. (2.10) into the equation of motion (Eq. (2.9b)) yields (Pain 1992; Thomson 1993):

$$\left(m\lambda^2 + c\lambda + k\right)Ae^{\lambda t} = 0 \tag{2.11}$$

Since $Ae^{\lambda t} \neq 0$ the following quadratic equation will be obtained:

$$m\lambda^2 + c\lambda + k = 0 \tag{2.12}$$

Hence:

$$\lambda_{1;2} = -\frac{c}{2m} \pm \sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}} \tag{2.13}$$

The properties λ_1 and λ_2 are the system's eigenvalues or poles (De Silva 2005).

For $\lambda_1 \neq \lambda_2$ the general solution to Eq. (2.9b) becomes:

$$y(t) = A_1 e^{\lambda_1 t} + A_2 e^{\lambda_2 t} \tag{2.14}$$

Since the differential equation (2.9b) is of second order, two separate values of A are allowed (Pain 1992). The arbitrary constants A_1 and A_2 are to be determined by the initial conditions.

When, however, $\lambda_1 = \lambda_2$ then there is only one solution for λ. To satisfy the initial conditions, and noticing that the differential equation is of second order, the general solution in this case becomes (Pain 1992; Thomson 1993):

$$y(t) = \left(A_1 + A_2 t\right)e^{-\left(\frac{c}{2m}\right)t} \tag{2.15}$$

where the arbitrary constants A_1 and A_2 are to be determined by the initial conditions.

Considering the square-root from Eq. (2.13), three types of damped motion can be recognised: overdamped, critically damped and underdamped.

Overdamped motion occurs if:

$$\left(\frac{c}{2m}\right)^2 > \frac{k}{m} \tag{2.16}$$

In this case the general solution (Eq. (2.14)) can be expressed as:

$$y(t) = e^{-\left(\frac{c}{2m}\right)t}\left(A_1 e^{\left(\sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}}\right)t} + A_2 e^{-\left(\sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}}\right)t}\right) \tag{2.17}$$

While overdamped motion can be described as exponential motion of the system straight backwards to the equilibrium position and resting there after being once deflected, the critically damped motion exhibits the same properties but additionally decays as fast as possible. Thus, in both cases no oscillatory motion can occur. The exponents of Eq. (2.17) are real numbers for the overdamped motion. The square-root of Eq. (2.13) amounts to zero for critical damping, which thus occurs when:

$$\left(\frac{c}{2m}\right)^2 = \frac{k}{m} \tag{2.18}$$

Hence, there is only one solution for λ with the general solution shown by Eq. (2.15).

The most occurring case in structural engineering is the underdamped motion. The system starts to oscillate. The displacements decay over time till the system potentially rests at the equilibrium position (Figure 2.3). This happens if:

$$\left(\frac{c}{2m}\right)^2 < \frac{k}{m} \tag{2.19}$$

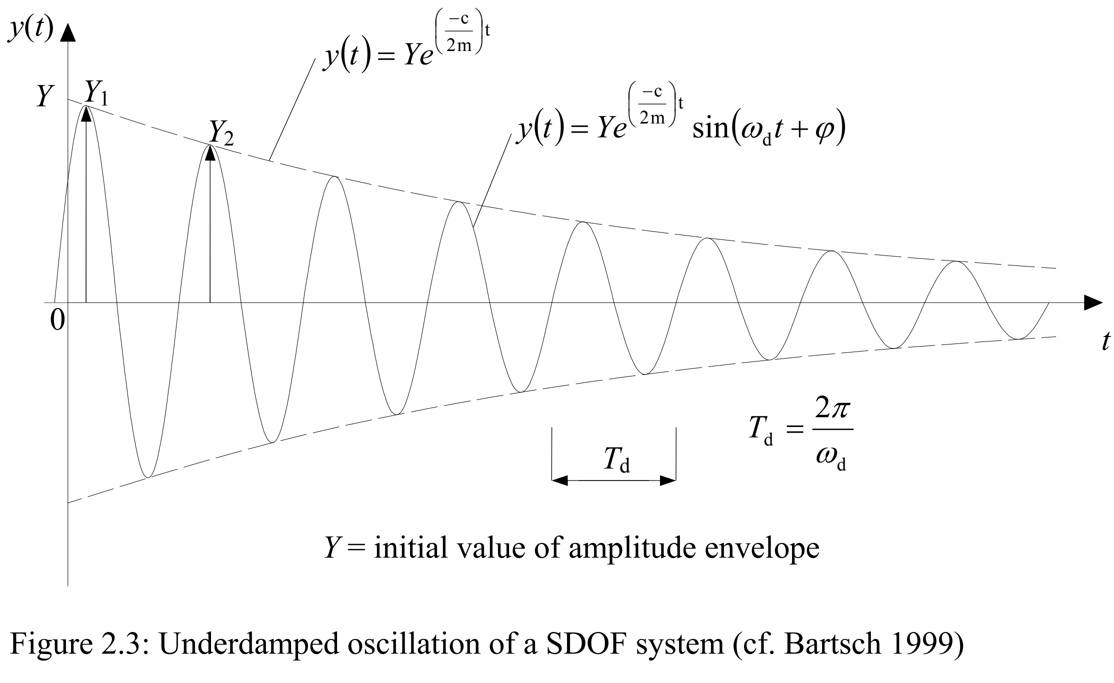


Figure 2.3: Underdamped oscillation of a SDOF system (cf. Bartsch 1999)

In this case the result under the square-root of Eq. (2.13) is negative. However, solutions can be determined by using complex numbers. Complex numbers are applied to express non real solutions, whereas imaginary units are added to the real numbers. The imaginary unit is symbolised by the property i. Definition: $i^2 = -1 \rightarrow i = \sqrt{-1}$. Thus, for this case, the equation of the property λ can be rewritten as:

$$\lambda_{1;2} = -\frac{c}{2m} \pm i\sqrt{\frac{k}{m} - \left(\frac{c}{2m}\right)^2} \tag{2.20}$$

In this case λ_1 and λ_2 are complex conjugates and the solution (Eq. (2.14)) can be expressed as (De Silva 2005):

$$y(t) = e^{-\left(\frac{c}{2m}\right)t}\left(A_1 e^{i\omega_d t} + A_2 e^{-i\omega_d t}\right) \tag{2.21}$$

As the physical system is real, the solution needs to be expressed in real terms. It is therefore concluded that A_1 and A_2 are complex conjugates. By using Euler's equations (De Silva 2005; Pain 1992)

$$e^{ix} = \cos x + i \sin x, \quad e^{-ix} = \cos x - i \sin x \tag{2.22}$$

alternative expressions of Eq. (2.21) can be formulated:

$$y(t) = e^{-\left(\frac{c}{2m}\right)t}\left(A_3 \sin \omega_d t + A_4 \cos \omega_d t\right) \tag{2.23}$$

with
$$A_3 = i(A_1 - A_2)$$
$$A_4 = A_1 + A_2$$

or
$$y(t) = \underbrace{Ye^{-\left(\frac{c}{2m}\right)t}}_{damping}\underbrace{\sin(\omega_d t + \varphi)}_{oscillation} \tag{2.24}$$

with
$$Y = \sqrt{A_3^2 + A_4^2},$$
$$\tan\varphi = \frac{A_4}{A_3}$$

The arbitrary constants A_3, A_4 and φ are to be determined by the initial conditions. The frequency of damped oscillation ω_d can be determined by the following formula (Bartsch 1999; Pain 1992):

$$\omega_{\mathrm{d}} = \sqrt{\frac{k}{m} - \left(\frac{c}{2m}\right)^2} = \sqrt{\frac{k}{m}} \cdot \sqrt{1-\zeta^2} = \omega\sqrt{1-\zeta^2} \tag{2.25}$$

The damping ratio ζ is a dimensionless value expressing the magnitude of damping:

$$\zeta = \frac{c}{c_{\mathrm{cr}}} \tag{2.26}$$

where c_{cr} is the critical damping coefficient in [kg/s] or [(Ns)/m]:

$$c_{\mathrm{cr}} = 2\sqrt{mk} \quad \text{or} \quad c_{\mathrm{cr}} = 2m\sqrt{\frac{k}{m}} \quad \text{or} \quad c_{\mathrm{cr}} = 2m\omega \tag{2.27}$$

Since the values of the damping ratio of timber floors usually amount to lower single percentage figures, there is only an infinitesimal small difference between ω and ω_d.

The amplitude is decreasing over time but the duration T_d of each cycle of damped oscillation stays the same constantly:

$$T_{\mathrm{d}} = \frac{2\pi}{\omega_{\mathrm{d}}} \quad [\mathrm{s}] \tag{2.28}$$

Damping of a system with $\zeta < 1$ can also be expressed by the dimensionless logarithmic decrement. A value of 0.1 means an amplitude decrease of 10% in any consecutive cycle. The relation of consecutive amplitudes is unchanging (Pain 1992):

$$\frac{Y_1}{Y_2} = \frac{Y_2}{Y_3} = \frac{Y_3}{Y_4} = \frac{Y_{\mathrm{n}}}{Y_{\mathrm{n+1}}} = e^{\frac{\mathrm{cT_d}}{2\mathrm{m}}} \tag{2.29}$$

Hence, the logarithmic decrement is:

$$\alpha = \ln\left(\frac{Y_{\mathrm{n}}}{Y_{\mathrm{n+1}}}\right) = \frac{cT_{\mathrm{d}}}{2m} \tag{2.30}$$

Deduced from this derivation the relation between the damping ratio and the logarithmic decrement can be described as follows (Thomson 1993):

$$\alpha = \frac{cT_{\mathrm{d}}}{2m} = \frac{c2\pi}{2m\omega_{\mathrm{d}}} = 2\pi\frac{c}{c_{\mathrm{cr}}\sqrt{1-\zeta^2}} = 2\pi\frac{\zeta}{\sqrt{1-\zeta^2}} \tag{2.31}$$

For very small values of ζ it can be approximated as:

$$\alpha \cong 2\pi\zeta \tag{2.32}$$

2.3 Forced damped vibration

So far, the motion of a SDOF system has been explained as it vibrates after being once deflected. Subsequently it will be shown how the system reacts if it is influenced by an external force. A harmonic excitation $F(t) = F_0 \sin \Omega t$ will be assumed. Thus, the differential equation, the equation of motion, is of the form (Pain 1992):

$$m\,\ddot{y}(t) + c\,\dot{y}(t) + k\,y(t) = F_0 \sin \Omega t \tag{2.33}$$

where F_0 is the excitation force in [N] and Ω is the forcing frequency in [rad/s].

The solution to Eq. (2.33) consists of a homogeneous and of a particular solution. The total solution is inhomogeneous:

$$y(t) = \underbrace{y_{\mathrm{h}}(t)}_{\text{damped free vibration response}} + \underbrace{y_{\mathrm{p}}(t)}_{\text{steady-state response}} \tag{2.34}$$

The homogeneous part results from the damped free vibration response. The particular part results from the external force and describes the steady-state response, which exists as long as the system is excited. Since the transient response, which decays to zero over time, has been covered before, the particular solution will be of interest subsequently. One option to analyse the particular part is (Thomson 1993):

$$y_{\mathrm{p}}(t) = Y \sin(\Omega t - \varphi) \tag{2.35}$$

in which Y is the steady-state amplitude:

$$Y = \frac{F_0}{\sqrt{\left(k - m\Omega^2\right)^2 + (c\Omega)^2}} = \frac{F}{k} \cdot \underbrace{\frac{1}{\sqrt{\left(1 - \eta^2\right)^2 + (2\zeta\eta)^2}}}_{\text{dynamic magnification factor}} \tag{2.36}$$

where η is the frequency ratio of forcing frequency over natural frequency:

$$\eta = \frac{\Omega}{\omega} \tag{2.37}$$

The particular solution - the steady-state solution - is obtained by substitution of Eq. (2.36) into Eq. (2.35):

$$y_{\mathrm{p}}(t) = \frac{F_0}{k} \cdot \underbrace{\frac{1}{\sqrt{\left(1-\eta^2\right)^2 + \left(2\zeta\eta\right)^2}}}_{\text{dynamic magnification factor}} \sin(\Omega t - \varphi) \tag{2.38}$$

where φ is the phase angle of the steady-state solution:

$$\tan\varphi = \frac{2\zeta\eta}{1-\eta^2} \tag{2.39}$$

From Eq. (2.36) it can be seen that the steady-state amplitude Y is equivalent to the static deflection F_0/k magnified by a fraction term. That fraction term is described as dynamic magnification factor (DMF), which is depending on the frequency ratio and on the damping ratio. The closer η to unity, the more the amplitude will increase and the more the amplitude is depending on the damping ratio as shown in Figure 2.4.

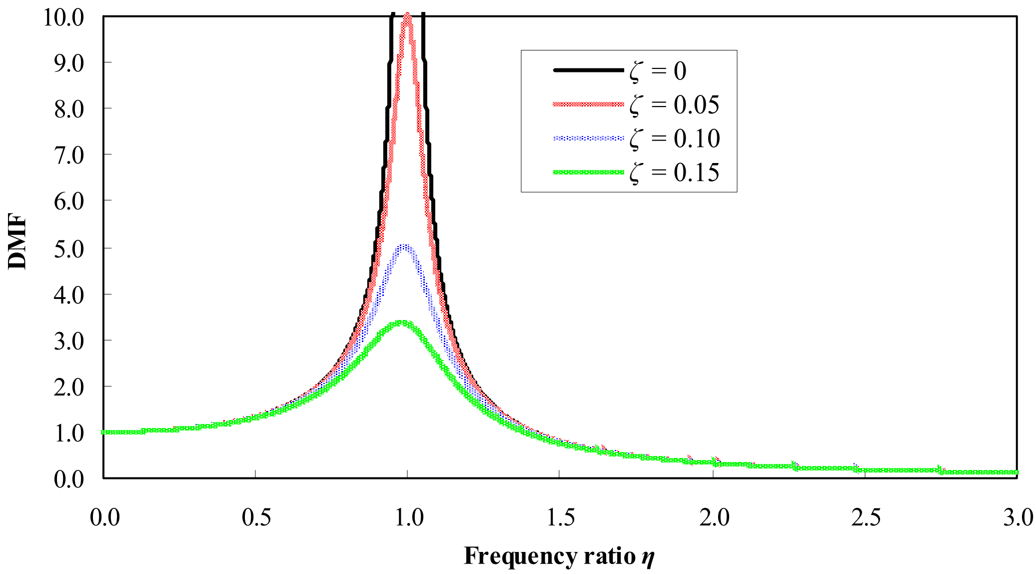


Figure 2.4: DMF for varied frequency ratios and damping ratios

The vibrational motion is less dependent on the excitation frequency if damping is high. For the case where natural frequencies are higher than the forcing frequency, an increasing natural frequency, by constant forcing frequency, leads to a lower magnification factor and hence to a lower displacement amplitude. It is thus favourable to assure that the natural frequency is much higher than the forcing frequency. From Figure 2.4 it is also obvious that for η lower than "1" the amplitude is decreasing with decreasing natural frequency for constant forcing frequency. In the case of flooring systems, however, it is hard to achieve most influential natural frequencies to be much less than forcing frequencies, which result from the low-frequency content of footfall impacts. The typical human walking rate is at 1.7 - 2.3 steps per second (Smith 2003).

For a value $\eta = 1$, the condition for resonance, the amplitude Y can be simplified, as comprehensible from Eq. (2.36), to:

$$Y = \frac{F}{c\Omega} = \frac{F}{2\zeta\, k} \tag{2.40}$$

The solution to the total response is:

$$y(t) = e^{-\zeta\omega t}\left(A_3 \sin\omega_{\mathrm{d}} t + A_4 \cos\omega_{\mathrm{d}} t\right) + \frac{F}{k} \cdot \frac{1}{\sqrt{\left(1-\eta^2\right)^2 + \left(2\zeta\eta\right)^2}} \sin\left(\Omega t - \varphi\right) \tag{2.41}$$

2.4 Analysis of real structures

Civil engineering structures have multiple degrees of freedom and are thus multi-degree-of-freedom (MDOF) systems. However, it is important to describe the properties of a SDOF system, as done above, since those for a "*MDOF system can always be represented as the linear superposition of a number of SDOF characteristics*" (Ewins 1984).

The MDOF system with n degrees of freedom is expressed in form of matrices and vectors:

$$\boldsymbol{M}\, \ddot{\boldsymbol{y}}(t) + \boldsymbol{C}\, \dot{\boldsymbol{y}}(t) + \boldsymbol{K}\, \boldsymbol{y}(t) = \boldsymbol{f}(t) \tag{2.42}$$

where $\boldsymbol{M}$ is the mass matrix, $\boldsymbol{C}$ is the damping matrix, $\boldsymbol{K}$ is the stiffness matrix, $\boldsymbol{f}(t)$ is the load vector, and $\boldsymbol{y}(t)$, $\dot{\boldsymbol{y}}(t)$ and $\ddot{\boldsymbol{y}}(t)$ are the displacement, velocity and acceleration vectors.

The dynamics of a system are described by its modal parameters: frequencies, damping and shapes. There are various numerical ways to manipulate the above expression (Eq. (2.42)) for the determination of the modal parameters. One method is to convert the continuous time model in Eq. (2.42) to a state space model in discrete time, which is the approach to modal analysis by the stochastic subspace identification (Section 4.5).

2.5 General considerations

Modal frequencies and damping are global properties; the properties that generate the mode shape are local ones. A mode shape is a deflection-pattern of relative displacements, which corresponds to a specific modal frequency. In the case of normal modes, described as standing waves with fixed nodal lines, all moving parts are vibrating in phase or 180° out of phase with each other (Døssing 1988b).

The physical displacement of a structure at a certain point is a combination of several modal shapes. For harmonic excitation with frequency close to a natural frequency of the system, the overall displacement will be largely dominated by the corresponding modal shape of that specific natural frequency (cf. Section 2.3), while "*random excitation tends to produce an arbitrary 'shuffling' of contributions from all the mode shapes*" (Døssing 1988b).

In literature, with respect to timber floor vibration serviceability, it is usually referred to the first order bending modes of the structures (Ohlsson 1982; Chui 1987). In this thesis, for a distinct identification, it is referred to the first principal first order (bending) mode as Mode (1,1), to the second principal first order (bending) mode as Mode (1,2), etc. In case of Mode (m,n), m denotes the number in longitudinal direction and n the number in transverse direction. Each normal mode is a combination of m and n. Thus, rectangular plate normal bending modes, apart from Mode (1,1), possess shapes with nodal lines, which are the lines with no modal displacement. Each mode possesses distinct locations with highest modal displacement, which are also termed as the anti-nodes.

3. Literature review

This literature review chapter is focusing on four main aspects: the sensitivity of humans to vibrations, the influence of floor variables on the dynamic performance, the control of dynamic floor behaviour, and the predictability of vibration parameters using analytical and numerical methods. Although the human response to floor vibrations was not a key aspect under consideration in the investigations presented in this thesis, it is required to provide a comprehensive review of this topic so as to identify parameters of major importance when examining dynamic responses of flooring systems.

3.1 Human sensitivity to vibrations

3.1.1 Research on human annoyance from floor vibrations

Reiher and Meister (1931, 1932) investigated human sensitivity to vibrations and shock. Ten test persons were exposed to vertical and horizontal vibrations while standing or lying on a platform. There were six classification categories from *not perceptible* to *very annoying/definitely dangerous in case of longer exposure*. It was shown that for steady-state vibrations where the test persons were exposed to the sinusoidal oscillation for about five minutes, the perception threshold was at a constant value of the product of amplitude (displacement) and frequency and thus at a constant vibration velocity. With respect to stronger vibrations, acceleration and its variation with time became more influential (Reiher and Meister 1931). From the tests with transient vibrations, which were caused by up to six impacts per second, the decay process due to an impact was found to hardly affect vibration perception if the damping decrement was equal to or larger than 0.1 (Reiher and Meister 1932).

Lenzen (1966) investigated the vibrational behaviour of composite steel joist - concrete slab floors and the human sensitivity to it, and observed that the main influencing factor on human beings from transient vibrations was the damping. Variation of amplitude and frequency showed little effect. If the vibration is damped to a negligible level in five cycles, humans would not be sensitive to it. If the system is still vibrating after twelve cycles, the human would react to it in the same way as to steady-state vibrations. Steady-state vibrations were assumed to usually not occur in buildings and transient vibrations to be of main interest.

Wiss and Parmelee (1974) investigated the perception of transient vibrations from a single-frequency component. Standing persons were exposed to vertical vibrations with varying frequency, peak amplitude and damping, which were then rated on a scale from 1 to 5, from imperceptible to severe respectively. The main study was conducted with 40 test persons. A test room with plan dimensions of $4.9\ \text{m} \times 8.5\ \text{m}$ was constructed and the vibrations were induced by a dynamic shaker, which was connected to the floor. Besides the transient vibrations, vibration signals with zero damping were also studied. After the tests, statistical analysis was carried out to identify relationships of various parameters. A number of statistical models were examined. To predict rating of damped vibrations, the following equation was proposed for the response rate *R*:

$$R = 5.08\left(\frac{f\,u_{\max}}{\zeta^{0.217}}\right)^{0.265} \tag{3.1}$$

where f is the frequency, $u_{\max}$ the peak displacement in inches and ζ the damping ratio. To predict the response from undamped vibration, the following equation was proposed:

$$R = 6.82\left(f\,u_{\max}\right)^{0.24} \tag{3.2}$$

Repeating some of the tests with 10% of the test persons seated showed that the vibration evaluation was not considerably affected whether the person was sitting or standing. The investigations furthermore showed that the product of frequency and displacement is a constant and that the transient vibrations of a certain frequency and peak displacement would be progressively less perceptible for an increase in damping.

Ohlsson (1982) performed subjective rating tests on timber floors and steel floors in laboratory (Section 3.2) with respect to springiness and vibrations in absolute terms. A relative rating of timber test floors with respect to a reference floor was also carried out. Usually 15 persons were asked to judge the floor performances individually. The tests on the laboratory timber floors showed that the reduction of span and the existence of ceiling would be positive and that the use of glue to fix the deck to the joists had little effect with respect to subjective vibration judgement. Heel impact tests indicated that anti-symmetrical modes contribute mostly to impact response and thus annoyance.

Ohlsson also noted that the resonances excited by heel impacts were interacting. Considering only the fundamental mode of vibration would not be sufficient for most timber floors since modes higher than the fundamental one could contribute significantly to annoyance. From the inverse of the spacing between adjacent natural frequencies it was found that the time between successive co-action of the lowest two modes was in the range of 0.25 s to 0.5 s, which was identified as an unsuitable interval regarding disturbance from impact loading. The co-action effect was described to be most severe if the damping coefficient of each co-acting mode is low so that high vibration amplitudes could be maintained within certain duration after the impact. Thus, the spacing between adjacent natural frequencies should amount to at least 5 Hz. The magnitudes of velocity responses in some tests indicated that higher first order modes should be considered when evaluating human response to vibrations as the initial peak velocity would be highly dependent on those higher modes. As reported by Ohlsson (1991), in case of typical timber floors, human annoyance would be governed by a composition of several frequency components of transient vibrations due to footfall excitation. According to ISO 2631 velocity reflects equal human discomfort for frequencies above 8 Hz.

Ohlsson (1982) reported four necessary distinctions regarding the perception of floor vibrations. The first aspect was whether the sensed floor vibration is self-generated from a single footstep or induced by other persons walking so as to distinguish between springiness and human induced floor vibrations. The second distinction was whether the motion is lightly or heavily damped. Damping was found to be very important regarding dynamic floor performances as the vibration duration would strongly influence annoyance. Ohlsson set up guidance by proposing a product of damping ratio and natural frequency as being the damping coefficient σ_0 (see Section 3.3). The value proposed to separate between light and heavy damping was $\sigma_0 = 0.4$ Hz, which was considered to be tentative. When $\sigma_0 \leq 0.4$ Hz, the floor would be regarded as lightly damped. When dealing with floor vibrations, damping with respect to time would be more suitable than the damping ratio, which expresses relative damping. It was assumed that at higher frequencies more cycles of transient vibrations would be required to cause vibration disturbance than at lower frequencies. The remaining two concerns would be to identify how many eigenmodes significantly participate in vibration induced by footfall and whether the floor is light-weight or heavy-weight. According to Ohlsson, a floor can be defined as light-weight if a human body on the floor considerably alters the modal properties of the structure.

Ohlsson concluded that for the enhancement of dynamic floor performances it should be mainly aimed at increasing the modal stiffness and the damping ratio in the case of continuous dynamic loading and the modal mass and the damping coefficient in the case of transient impulsive loading. Hence, raising damping or stiffness would never result in negative effects. Raising mass would only be positive regarding transient vibrations but still with a negative effect on the fundamental frequency. Raising transverse stiffness would be especially efficient to reduce springiness and problems from impact vibrations and could also reduce problems from continuous vibrations. An increase in fundamental frequency would be an efficient measure for enhancing floor serviceability but a further option would be to increase the frequencies of higher vibration modes.

Chui (1987) carried out field tests on six floors with different acceptability ratings (see Sections 3.2). The vibration response was determined based on heel-drop impact tests, in which a person in floor centre experienced the floor response after performing a heel-drop excitation. It was found that root-mean-square acceleration (r.m.s. acceleration) encountered in practice would lie between 0.1 - 0.8 m/s^2 and that human perception of vibration could be related to the different magnitudes of r.m.s. acceleration as presented in Figure 3.1. While the complexity of defining threshold levels with respect to human sensitivity was still highlighted, it was found that frequency-weighted r.m.s. acceleration $a_{\mathrm{w,rms}}$ for design should be less than 0.45 m/s^2.

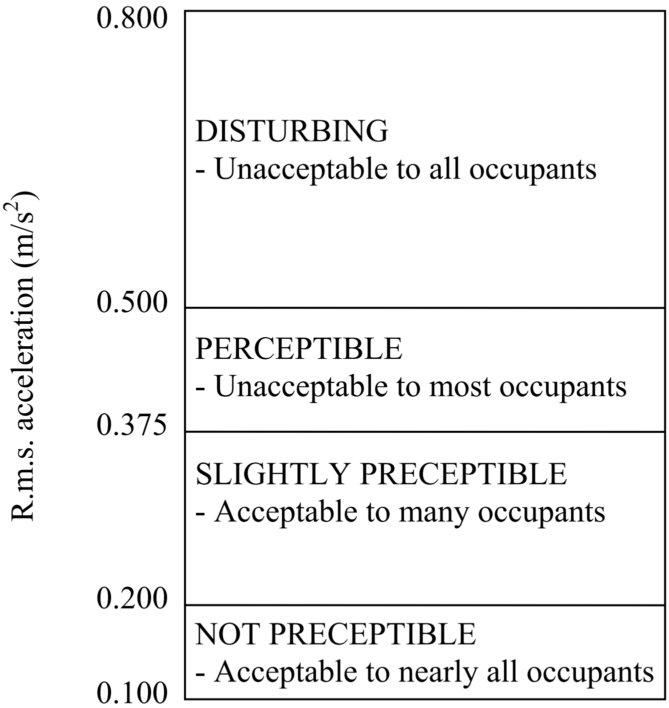


Figure 3.1: Human response to different magnitudes of r.m.s. acceleration according to Chui (1987)

Hu and Tardiff (2000) investigated the effect of the installation of strongback or I-joist blocking on subjective vibration classifications of wood truss and wood I-joist floors respectively (see also Section 3.2). The floor performance was rated by 20 persons individually. A person was asked to rate the performance when walking over the floor and while sitting on a chair in floor centre when another person excited the floor by walking. Generally all floors showed enhanced performance with respect to the subjective rating when the transverse stiffeners were installed. Initial velocities and static deflections indicated better vibrational performance for the floor with two strongback elements at mid-span compared to one strongback at that location, whereas the subjective rating showed a different trend. Overall results indicated that strongback or I-joist blocking should be installed at a spacing of 2 m. It was identified that more information would be still required to further quantify the effect of transverse stiffeners on the floor performances.

From data analysis of field tests on floors and vibration ratings of occupants, Alvis (2001) found no correlation of human perception with either peak acceleration or some filtered peak acceleration or r.m.s. acceleration or fundamental natural frequency or product of fundamental frequency and peak acceleration. The fact that the acceptability of vibrations was not correlating well with the measured data was partially attributed to different annoying frequencies at different floor locations, and some floor locations would be more sensitive to contributions from higher modes. It was therefore suggested to rate floor vibration at different floor areas. Alvis also concluded that FEM analysis instead of hand calculations should be conducted for natural frequency prediction if a mode other than the first one is contributing most to annoying vibration.

Ljunggren (2006) and Ljunggren et al. (2007) investigated the influence of multi-frequency components on human vibration perception. A test person was sitting on a chair, which was placed on the top of a relatively stiff wooden plate with resonance frequencies outside the frequency range of interest. The wooden plate was part of a motion simulator including an electromagnetic shaker. There were 15 test persons exposed to single- and dual-frequency components individually to evaluate the disturbance of vibration (Ljunggren et al. 2007). The first frequency component was selected to be at 8 Hz with constant amplitude. The second component was composed of one of five frequencies, each with five different amplitudes. It was found that the sensitivity of humans to vibrations was considerably increased when a frequency component was added to the first component of 8 Hz, in particular when the two frequencies were closely spaced. Furthermore, people tended to be more disturbed for an increase in the amplitude of the second component but less disturbed for an increase in its frequency. It was concluded that consideration of higher natural frequencies is required in design as the vibration perception could be considerably influenced by them.

The same test platform was used with 15 test persons rating vibration responses on an annoyance and an acceptance scale when they were exposed to single-frequency components at different frequencies and to multiple frequency-components of varying combinations. The amplitude of each frequency component was set to a distinct value, selecting from two different levels. This resulted in a total of 44 signals used (Ljunggren 2006). The results of the study matched the trend described above. It was highlighted that the sensed disturbance was not necessarily decreasing when the frequencies increased. This was explained by low frequency-spacing inducing beating effects in the summed signal. It was furthermore identified that the frequency-weighting method of ISO 2631-2 (2003) performs well for rating annoyance in the case of a single sinusoidal but has a lower accuracy with respect to a set of discrete frequency components. To predict the disturbance sensed by people in domestic and office buildings, models were suggested for the case of vibrations with single- and multiple-frequency components (Figure 3.2). The models were determined by multiple regression analyses, testing the statistical significance of the total amplitude, the fundamental frequency and the frequency separation for varied models.

Sinusoidal case:

$$Annoyance = -1.26 + 0.39 \cdot Weighted\ total\ amplitude$$

Multiple frequency case:

$$Annoyance = -3.17 + 0.43 \cdot Weighted\ total\ amplitude + 0.24 \cdot Fundamental\ \ frequency$$

Where *amplitude* is given in mm/s^2 rms and *frequency* in Hz.
Frequency weighting, W_m, according to ISO 2631-2 2003.

Interpretation: If $Annoyance \leq 4$, the floor is acceptable
If $Annoyance > 4$, the floor is unacceptable

Figure 3.2: Models to predict human annoyance by Ljunggren (2006)

Bernard (2008) examined flooring systems with respect to subjective rating of vibration performance. It was reported that none of the investigated structural modifications (see Section 3.2) led to enhanced vibrational behaviour apart from reducing joist depth or inserting rubber material. The floors with lower joist depth were found to have reduced rigidity but were more comfortable to walking on. Blocking was found to enhance load distribution between joists, but it would not enhance vibration behaviour. However, the vibrational performances of the floors were in majority found not to disturb with respect to bodily oscillation but, in most cases, with respect to drumminess (thump-like response being primarily audible) and shake (higher frequency response suggesting rapid oscillation of floor components) due to footfall. Main conclusions from the study were that many design features and proposed measures for enhancing dynamic serviceability of light-weight engineered timber floors would be highly ineffective, and that design criteria with respect to drumminess and shaking would be required. It was found that retaining a damping ratio of 5% or more might result in acceptable vibration performance regarding shaking, and that minimum limits for damping need to be established.

3.1.2 Standards for evaluating human response to vibration

To evaluate human sensitivity to vibrations, the international standard ISO 2631 (currently ISO 2631-1:1997 and ISO 2631-2:2003) is often referred to. It defines methods of filtering vibration magnitudes with respect to frequency and usually the direction of exposure. The perception of vibration can vary whether a person is standing/seated or lying and is also frequency-dependent. Therefore, frequency-weighting functions were established for different directions of the coordinate system, whose orientation moves with the human body (see ISO 2631-1). Frequency-weighting curves defined in ISO 2631-1 are shown in Figure 3.3.

Part 2 of ISO 2631 specifically deals with vibration in buildings. Although acceptable vibration magnitudes are not established, it provides tentative guidance. Base curves were determined to quantify the sensitivity of humans to vibration responses in terms of acceleration or velocity responses against frequency as shown in Figure 3.4 (ISO 2631-2:1989). As the human tolerance to vibrations is also dependent on factors such as location, type of excitation and time of the day, multiplication factors depending on the appropriate situation are to be applied to the base curves to specify satisfactory vibration levels (Table 3.1). It was stated that no adverse comments or complaints were in general reported for magnitudes of acceleration or velocity below the specified base curves, which should, however, not mean that annoyance is expected at higher vibration levels.

British Standard BS 6472 (1992) is based on ISO 2631-2 (1989) and contains base curves, which are generally the same. Also the multiplication factors to be applied to the base curves are the same if continuous vibration is considered, but some modified factors are given for the case of transient/impulsive vibrations. The standard gives further guidance on assessing the likelihood of adverse comments using some vibration dose value (VDV).

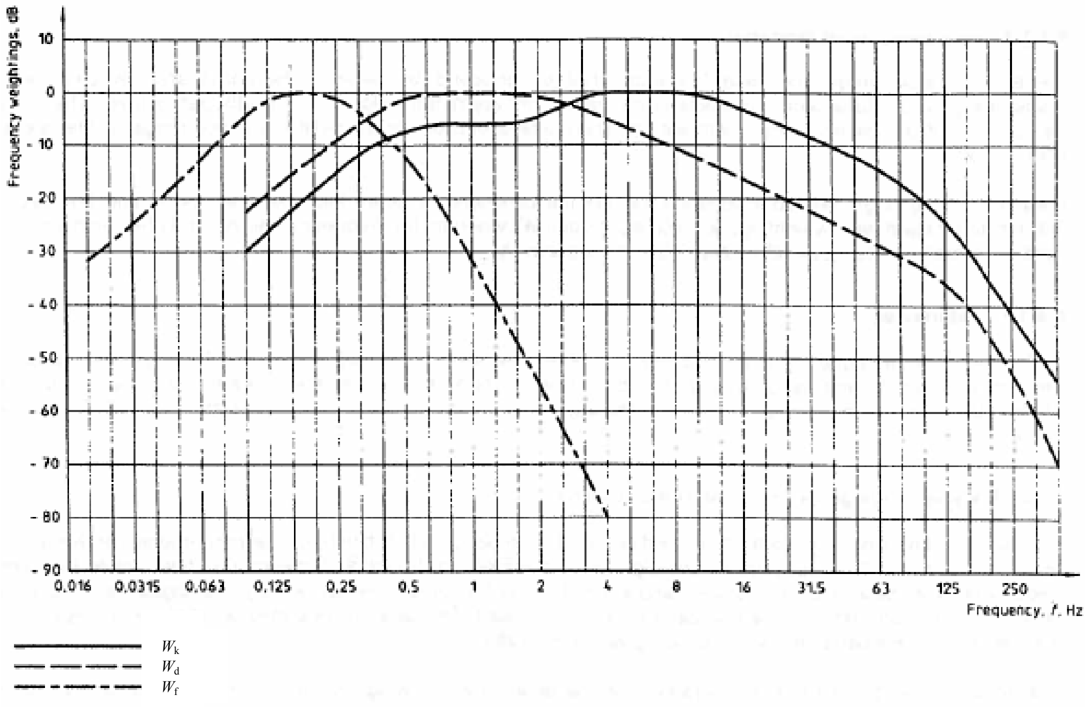
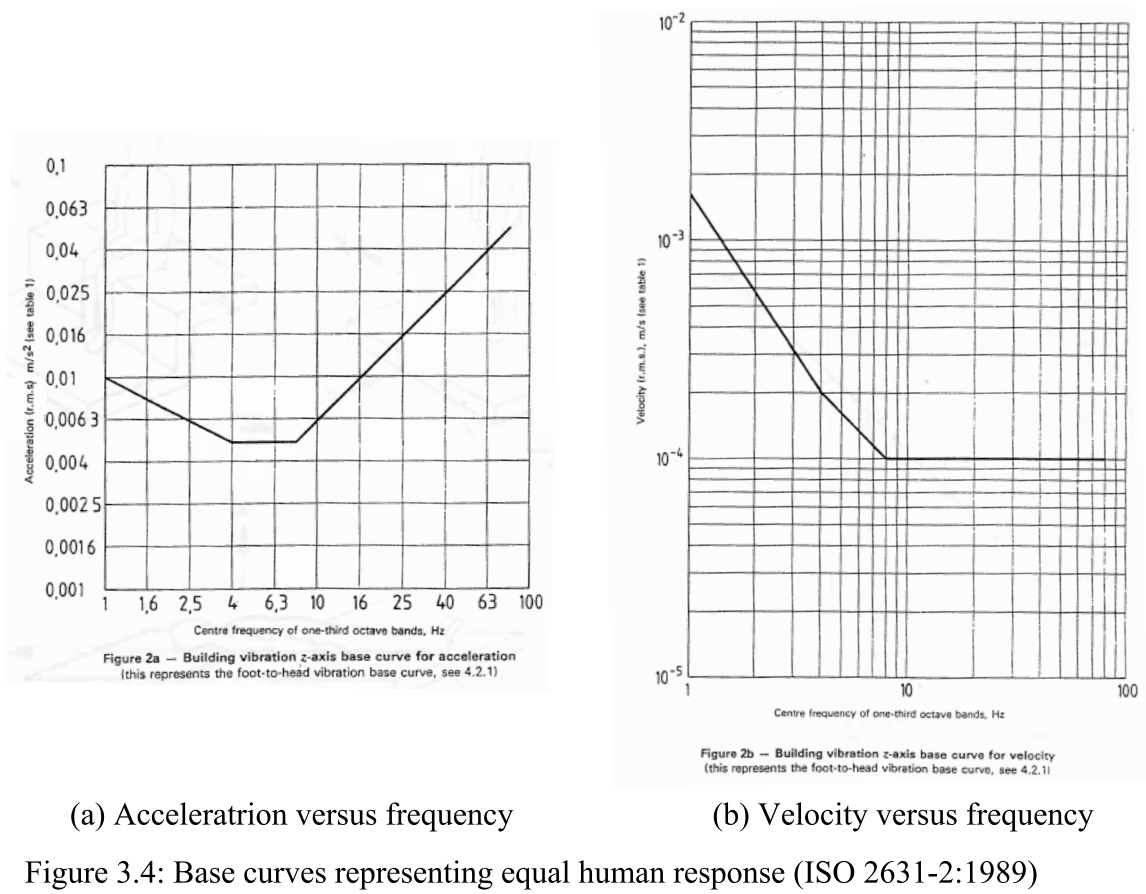


Figure 3.3: Frequency-weighting curves for principal weightings with W_k applied to z-axis vibration (ISO 2631-1:1997)



(a) Acceleratrion versus frequency (b) Velocity versus frequency

Figure 3.4: Base curves representing equal human response (ISO 2631-2:1989)

Table 3.1: Multiplying factors used with base curves to specify satisfactory magnitudes of human response to building vibrations (ISO 2631-2:1989)

Place	Time	Continuous or intermittent vibration	Transient vibration excitation with several occurrences per day
Critical working areas	Day Night	1	1
Residential buildings	Day	2 to 4	30 to 90
	Night	1.4	1.4 to 20
Office	Day Night	4	60 to 128
Workshop	Day Night	8	90 to 128

The revised versions of ISO 2631-2 published in 2003 and BS 6472 published in two parts (BS 6472-1; BS 6472-2) in 2008 contain significant changes. In both standards the base curves and corresponding multiplication factors for determining levels of approximately equal human response on annoyance were withdrawn. The possible range of threshold values would be too widespread to be reproduced (ISO 2631-2). This was also highlighted by Ljunggren (2006). In BS 6472-1 the approach for estimating the likelihood of adverse comments was solely based on the VDV, which "*defines a relationship that yields a consistent assessment of continuous, intermittent, occasional and impulsive vibration and correlates well with subjective response.*" To assess the likelihood of adverse comments within residential buildings, a range of VDVs are provided for each classification category (Table 3.2). Those for the day time are the same as given in the superseded standard whereas those for the night time used to be distinct values. Furthermore, multiplying factors are provided to account for other types of rooms, as shown by the note of Table 3.2.

Both revised standards (ISO 2631-2 and BS 6472) present modified methods for frequency-weighting vibration responses to account for frequency-dependent variation of human sensitivity, whereas the methods in the two standards differ. The current version of ISO 2631-2 (2003) contains a weighting function, which is applied to vibration responses where it is not required to define the posture of a person. If posture of a person is defined, however, the individual frequency-weighting functions defined in ISO 2631-1 (1997) can be used. In BS 6472-1 the frequency-weighting function for vertical vibration was adjusted so as to take higher frequency components more into consideration (Figure 3.5). It states that the "*difference could be a factor of 1.4 lower for vibrations that are predominantly at the lowest frequency, or could be a factor up to two higher for vibrations with dominant components at the top end of the frequency range*." The used frequency-weighting function for vertical vibration, W_b, is defined in BS 6841 (1987). Although the adopted weighting function accounts more for higher frequency components than the one in the superseded standard, "*at and just above the threshold of perception it seems that even W_b gives insufficient weight to vibration at the higher frequencies of the range considered*" although "*W_b is the most appropriate frequency weighting network for use with vertical vibration when the levels of vibration are clearly above the threshold of perception.*" The human oriented coordinate system was exchanged by a geocentric one.

Table 3.2: Ranges of VDVs in m/s$^{1.75}$ for different situations (BS 6472-1:2008)

Place and time	Low probability of adverse comment	Adverse comment possible	Adverse comment probable
Residential buildings 16 h day	0.2 to 0.4	0.4 to 0.8	0.8 to 1.6
Residential buildings 8 h night	0.1 to 0.2	0.2 to 0.4	0.4 to 0.8

"*Note: For offices and workshops, multiplying factors of 2 and 4 respectively should be applied to the above vibration dose value ranges for a 16 h day.*" (BS 6472-1:2008)

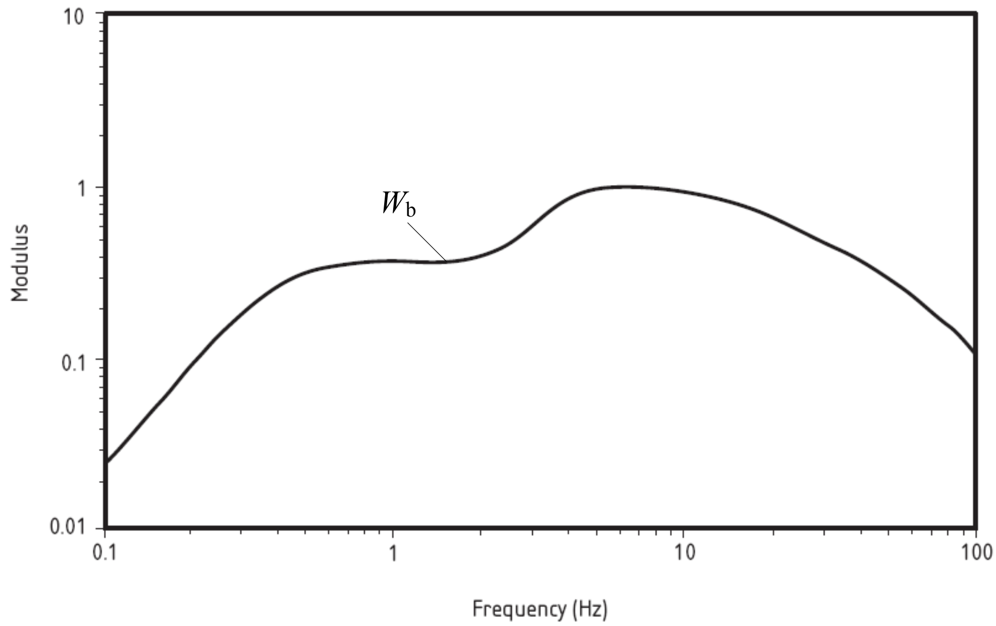


Figure 3.5: Weighting curve modulus for vertical acceleration (BS 6472-1:2008)

3.2 Vibrational floor performances

Lenzen (1966) found that the presence of occupants increased the damping of composite steel joist - concrete slab floors. However, other kinds of loads would not raise the damping values. The damping of a floor was strongly decreased when being loaded with concrete cylinders.

Rainer and Pernica (1981) investigated the effect of two impulse excitation methods and two continuous methods on the modal damping ratios of a large-scale floor fabricated with open-web steel joists and concrete slab. They confirmed that humans on a flooring structure add damping to the system so that damping ratios obtained from heel-drop tests usually were higher than those measured on bare floors. In addition, the location of the person performing the heel-drop test affected the damping ratio.

Ohlsson (1982) studied the dynamic performance of initially eight timber flooring systems in the laboratory and four on site. The laboratory floors, constructed with five solid timber beams of 45 mm × 220 mm at a spacing of 600 mm, which resulted in a floor width of 2.4 m, were supported at the joist ends (free edges). The deck was 22 mm thick particleboard, which was connected to the joists with wood screws for six test floors and with a combination of screws and glue for the other two floors. The spans of the floors were 5.0 m (two floors), 4.25 m (two floors) and 3.5 m (four floors), respectively. Half of the floors had 13 mm thick ceiling, and the other half had no ceiling. In addition to these floors, a reference floor was constructed to serve especially for comparisons when investigating the human sensitivity (see Section 3.1). The construction parameters of the reference floor were similar to those of the other floors with a span of 3.8 m and the deck fixed to the joists by glue and screws. In addition, wooden boards of 35 mm × 140 mm were fixed to the bottom of the joists in transverse direction at each third-span of the floor. A hammer impact excitation was first adopted. Acceleration response in vertical direction on the top of the floor surface was measured during vibration. For six flooring systems the measurements were taken at 15 points distributed over the surface and for two floors at 17 points along the central line in transverse direction. The vertical deflections at the floor centre under static loading were also measured. Load levels of 1 kN, 2 kN, 3 kN and 4 kN were applied, respectively. Ohlsson determined the point flexibility (displacement at floor centre divided by the applied load at that point) and the transverse flexibility (displacement at floor centre reduced by the average displacement at mid-span of the edge joists and divided by the load at floor centre).

The tested field floors were part of timber framed one-family houses. Two of the floors were in furnished houses and two in empty houses. The floor of a whole storey was defined as the system and had wooden beams over two spans. Outer and central basement walls served as supports. Most floor areas were covered with 22 mm thick particleboard. Cross-bridging was used at mid-span. The number of loading and measurement points varied with floors. The loading points were, however, always located at mid-span. Mid-span deflections of the floors were measured under the static load of the test conductor which was applied closely to the measurement point.

From the tests on the laboratory floors, first- and second-order modes were identified in the frequency range of up to 80 Hz. Additional modes were classified as plate modes or, if they could not be fully described, as false modes. The two lowest natural frequencies for one flooring system appeared to coincide. From this experimental work, Ohlsson concluded that, apart from the third mode in some cases, principal first-order modes were identified for all test floors and some modes other than the principal ones existed of which some were plate modes. Ohlsson further concluded that natural frequencies are mostly closely spaced and that fundamental natural frequencies lie in the range of 12 - 28 Hz. Damping ratios were found to be in the range of 0.5% to 1.3%, hardly exceeding 1.5% for any of the first five first order principal modes. Furthermore, the damping variation was small whether the deck was glued or screwed to the joists. From deflection tests it was noted that the addition of ceiling board and the reduction of floor span considerably affected the point stiffness. With respect to transverse flexibility, the major part of the force was carried by the loaded joist and the edge joists sustained little deflections.

Ohlsson repeated tests on the laboratory timber floors with two types of excitation: random walking (on all nine floors) and heel drop impact (on seven floors) to identify which modes would be excited by human footfall and to determine typical vibration acceleration magnitudes. The first four principal modes were well excited by walking and several modes by heel impacts. These tests indicated that the frequency range of interest could be from 0 to 50 Hz. Tests from footfall excitation on the field timber floors by random walking showed that there were no major contributions from frequencies above 40 Hz in the evaluated acceleration spectral densities.

Ohlsson also tested some floors with steel as main structural components. One of the most interesting findings was that the method "random running", which was not used on the timber floors, more easily excited higher first order modes than "random walking".

Tests on a steel-concrete floor showed little difference in acceleration response levels in the low-frequency range whether the person exciting the floor was wearing clogs or socks. Acceleration response magnitudes were higher for the frequency range of 20 - 40 Hz when the test person was walking with clogs.

Comparison of the fundamental natural frequencies of laboratory and field floors indicated that, in this regard, laboratory floors could be considered representative for real floors. With respect to higher modes, it was noted that a direct comparison may not be adequate as the modes to be compared should have the same geometrical wave-length in y-direction (transverse direction). The damping ratio of the fundamental mode was 0.9% on average for the laboratory floors and about 3.4% for the field floors. Therefore, damping of laboratory floors would not exactly represent the damping of real floors (see Section 3.5 and Section 8.4). It was, however, emphasised that the damping of laboratory floors, with ideal support conditions, could serve as a lower limit of damping that could be expected for corresponding floors in real buildings, and that damping of laboratory floors was determined with a higher degree of accuracy. Relatively low scatter for the transverse flexibility values was noted for the field test floors, and their transverse flexibility was found to be about 55% of the corresponding laboratory floors. Taking into account the collective effects of different boundary conditions, existence of cross-bridging and spare battens of the field floors, the obtained reduction was considered disappointing so cross-bridging would not be very efficient.

Ohlsson also conducted tests on two additional laboratory floors with a total of four configurations since damping and transverse stiffness were identified as important parameters. The two floors of 4.3 m span were constructed with glulam beams of 53 mm × 225 mm and 22 mm thick particleboard deck. For one of the systems screws were adopted to connect the deck to the joists, and the other one had viscoelastic layer strips between the joists and deck. Comparison of the results of both configurations showed that the use of the viscoelastic layer was efficient with respect to the damping of lower modes, e.g. 3.5% (80 days after application) instead of 1% for the fundamental mode. The presented results showed that the natural frequencies of the floor with viscoelastic layer had decreased. The floor without viscoelastic layer was further modified. An alternative method to cross-bridging was developed for transversely stiffening the floors. The cross beam consisted of top flange sections, which were fixed to the underside of the deck by using screws and glue, and a web at one side, both inserted between the longitudinal joists, and a continuous bottom flange. The first two natural frequencies were not affected by this measure but the frequencies of higher modes increased considerably. Then, a viscoelastic layer was placed at the interface of top flange and web of the transverse beam and tests were repeated. The results showed little effect on the damping of the first two modes but increased damping for the next two modes. The frequencies of all four presented modes were decreased with notable influence on the third and fourth mode in comparison with the same flooring system without viscoelastic layer.

Chui (1987) performed sensitivity studies regarding vibrational behaviour of a timber floor with varied parameters using his mathematical model (see Section 3.4) to propose construction techniques for enhanced vibrational performance of timber flooring systems. He also investigated the influence of several varied construction parameters experimentally.

Six timber floors based on the then state-of-the-art UK practice were constructed for the experimental studies. Solid timber joists were used for the structures, which were decked with chipboard, plywood or softwood boarding. Deck and joists were connected by commonly used nails and in one case by adhesives. The span varied between 3.6 m and 3.92 m. The six floors were supported along two sides. They were retested after construction specifications were modified, resulting in a total of twelve examined floors. The effect of strutting between joists, dead loads, fastening methods (denser nail spacing, elastomeric adhesive instead of nails), plasterboard ceiling, decking material, end fixity, internal and edge supports, human presence and varied joist spacing were systematically studied.

From the comparisons of vibration characteristics of two floors with joist spacing of 600 mm and 400 mm, Chui observed an increase of about 8% in the frequency corresponding to the fundamental mode but decreases in frequencies corresponding to modes higher than the second one, with a closer spacing between frequencies of adjacent modes if lowering the joist spacing. The damping ratios were increasing on average.

For two flooring systems tested with and without solid blocking, which was installed at the third-spans of the floors, the frequencies corresponding to the first two modes only slightly decreased and the frequencies of higher modes increased, resulting in more separated natural frequencies. The variation in damping was found to be inconclusive when blocking was installed.

For a flooring system tested with and without an imposed line load of 100 kg (0.98 kN) applied along the free edges, the frequencies of the two lowest modes decreased while the frequency spacing became wider and the damping ratios increased for the addition of load. It was also observed that the added line load reduced the modal displacements at unsupported edges and that the second mode would have been eliminated.

Reducing the nail spacing from 250 mm to 125 mm for the decking-to-joist connection of one flooring structure slightly increased natural frequencies, but the results regarding damping capacity were inconclusive. When the screws connecting deck and joists were replaced by elastomeric glue for one flooring system, a slight increase in natural frequencies was observed but little effect on the mean damping ratio. The damping varied differently for the individual modes.

The addition of plasterboard ceiling to one of the test floors was found to increase the damping capacity by about 18% on average, to lower the first two natural frequencies and to raise the higher ones. The study on the decking material including 22 mm thick chipboard, 15 mm thick plywood or 19 mm thick softwood boarding showed that the floor with the stiffer plywood deck possessed higher natural frequencies than the floors with the other decking types, which performed similarly. Damping ratios for the floor decked with chipboard were higher in general.

The effect of end fixity was examined by doubling the tightening torque on the bolts used for clamping the joist ends. The natural frequencies were raised by about 1% and damping was little affected. The effect of varied support conditions was also studied on the originally two-side supported floor by introducing an additional support under the middle joist, additional supports under the outer joists and then a combination of both. It was noted that additional supports generally raised the fundamental frequency, in particular if a middle support was added, and made the lower vibration modes more dominating. Supporting the outer floor joists caused adjacent natural frequencies to be more separated whereas the opposite was observed for adding an internal support. Some modes almost overlapped each other when only the middle joist was supported additionally. For all cases with extra supports, the damping was increased compared to the original floor. Another two-side supported flooring system was also re-tested after the outer joists were supported. The obtained results confirmed considerable increases in natural frequencies and damping for this measure.

Impact tests on flooring systems were repeated, exchanging the hammer excitation to heel-drop excitation, which was performed by a person on the floor. The presence of the person on the floor slightly reduced natural frequencies, but the damping ratios obtained were higher. Chui reported a damping ratio of about 1.5% as an average of all damping ratios obtained from the vibration tests on the investigated floors.

The theoretical sensitivity study included the effects of floor dimension, modulus of elasticity (MoE) of joist, decking stiffness, load-slip modulus of the decking-to-joist connection, decking gaps, between-joist strutting, static loading, damping, duration and magnitude of impact and edge support conditions. The reference floor had a size of 3.6 m × 3.6 m, constructed with 22 mm thick decking connected to seven joists of 0.05 m × 0.2 m, and a damping ratio of 4% was assumed for analysis. The floor with free edge joists was simply supported at the joist ends. The floor response was mainly measured in floor centre, which was also the location for the excitation.

An increase in span largely lowered natural frequencies and raised r.m.s. acceleration while the rate of increase in acceleration increased accordingly. The spacing between natural frequencies was little influenced. A slight reduction in the fundamental frequency and a narrower spacing between adjacent natural frequencies were found for increasing the floor width. The r.m.s. acceleration was then lowered.

The MoE of joists was varied between 4000 and 14000 N/mm^2 and little influence on vibration performance was found, especially at the upper end of that range. The increase of MoE would however narrow the separation between neighbouring natural frequencies. Still, a reduction of 27% in r.m.s. acceleration and an increase of 13% in the fundamental frequency were obtained by doubling the MoE value of 6000 N/mm^2.

For raising the MoE of the decking in longitudinal direction gradually from 2000 to 14000 N/mm^2, no significant effect on the vibrational performance was obtained. For the same measure in the transverse direction, the fundamental frequency was little affected but the sensitivity increased with the mode number. When increasing the MoE from the lowest examined value to the highest one, the natural frequency increased by 4% for the second mode and by 75% for the fifth mode. The r.m.s. acceleration was reduced by about 28%.

The vibrational performance of a timber floor was found to be theoretically sensitive to changes in the load-slip modulus of decking-to-joist connections, but in practice a significant increase in the modulus would be needed to yield noteworthy effects. It was further identified that also decking gaps can notably influence vibration characteristics by increasing the r.m.s. acceleration and lowering the natural frequencies.

The use of transverse stiffeners such as blocking or strutting reduced the r.m.s. acceleration responses and raised higher natural frequencies. The higher the number of blocking rows, the stronger was the effect. Up to five rows of blocking were considered. Raising uniformly distributed load or point load lowered natural frequencies. The variation in r.m.s. acceleration was dependent on the loading case.

For increasing the damping ratio, it was found that the decrease rate of the r.m.s. acceleration became slower and that the r.m.s. acceleration became little sensitive to increases in the damping ratio above 3%. The r.m.s. acceleration was found to increase when raising the peak magnitude of an impact force and to generally decrease when raising the impact duration. For increases at very short impact durations, the r.m.s. acceleration increased.

Supporting the outer floor joists was found to raise the frequency spacing between adjacent modes while the r.m.s. acceleration was little affected at floor centre and slightly reduced at the mid-span of the joists adjacent to the central one.

Based on the sensitivity study it was recommended to support the outer floor joists so as to raise natural frequencies and to lower mean r.m.s. acceleration. Strutting was advised to be applied at 1 m intervals, which would yield similar effects except that the fundamental frequency would not change. It was furthermore recommended to span the floor in the shorter direction so as to benefit in particular the fundamental frequency and the r.m.s. acceleration. Orthotropic decking panels should be placed with the stiffer direction perpendicular to the joists and should have a high bending stiffness in the across-joist direction. Chui also suggested gluing the joints between decking panels, which increases the degree of composite action, so as to raise the stiffness of timber floors.

Smith and Chui (1988a) recommended 3% to be used as the damping ratio for light-weight floors. They found a material damping ratio of about 1% for solid timber (Canadian White Spruce) and concluded that the total damping of a floor would increase with increasing mass, justifying the use of the proposed damping ratio (Chui and Smith 1989). By summarising varied terms and expressions for damping, they described the dependence of accurate damping estimates on testing and analysing methods. Chui and Smith suggested use of the viscous damping ratio to express the damping magnitude of timber structures.

Hu (1992) conducted vibration tests on six timber I-joist floors with different structural configurations, focusing on the effect of varied support conditions and spans to validate the proposed mathematical model (see Section 3.4). One flooring system was re-tested with four different load cases. The damping ratios corresponding to the fundamental vibration modes were found to lie in the range of 2.9% to 7.8%. Damping ratios of higher modes lay between 1.7% and 5.4%. The high damping ratios of some fundamental modes were thought to be caused by the testing procedure so that it was finally judged that damping ratios of the tested I-joist floors would lie between 2% and 4%. The addition of mass was found to raise the damping of the system. The variation in damping was further dependent on the magnitude and location of the imposed mass. Also the type of imposed mass would influence damping. The added mass reduced the natural frequencies of the fundamental modes and mainly also of the second modes, but the frequencies corresponding to higher modes were mainly increased. The variations in natural frequencies and damping due to other structural configurations were not directly studied. The results show, however, that a reduction in span raised all natural frequencies, but the effects on damping were inconclusive. Supporting all four edges instead of only the joist ends sometimes lowered the fundamental frequencies but usually raised the higher ones. The change in damping was not conclusive but with a tendency to increase.

The effectiveness of strongback and wood I-joist blocking on the dynamic performance of floors with engineered timber joists (wood floor trusses, I-joists) was investigated by Hu and Tardif (2000). The wood truss floor had dimensions of 5.9 m × 4.9 m and was decked with 15.5 mm thick plywood. Four configurations were tested, one without strongback, and the others with either one strongback installed at mid-span or two strongback elements at mid-span or spaced at about two metres. The I-joist floor had a size of 4.9 m × 4.9 m and was also decked with 15.5 mm thick plywood. Tests were performed on the floor without I-joist blocking and with one row of I-joist blocking installed at mid-span. It was found that the installation of the transverse stiffeners significantly lowered the static deflections under 1 kN point load and reduced also dynamic responses in floor centre. For the truss floor, the fundamental frequency changed little whereas higher mode frequencies were clearly raised. However, for the I-joist floors all natural frequencies were almost unchanged, and the damping varied little. Due to the installation of strongback at mid-span, the floor centre was no longer the weakest area. The highest initial velocities and static deflections were instead found at locations about 1 m from the mid-span on either side.

The truss floor structure without strongback was also used to investigate the influence of partition walls on the dynamic performance, as presented by Hu and Tardif (1999). Three different types of partition were placed on the floor separately. In one case the wall was placed over a joist at about a third of the width; in another case the wall run perpendicular to the joists at a third of the span; and in the third case an L-shape partition was encompassing a quarter of the floor. It was found that the addition of a partition wall lowered initial velocities and static deflections in most cases. In some cases the deflection decreased considerably. The fundamental frequency generally increased, except for the floor with the L-shape partition where the fundamental frequency decreased. The corresponding damping ratios were almost the same except again the floor with the L-shape partition which had considerably lowered damping. The number of modes below 40 Hz was unchanged (partition parallel to joists) or reduced by one. The spacing between the frequencies of the two lowest modes was varying little, except the floor with the partition perpendicular to the span direction with notably higher separation of the two lowest modal frequencies. It was concluded that a partition wall perpendicular to the joists enhances the transverse stiffness while adding a line mass, which would collectively enhance vibration performance (e.g. raised fundamental frequency, reduced initial velocity and reduced point load deflection). Introducing a partition parallel to the floor joists would reinforce the joist at which it is located and add a line mass. This would only enhance the vibration performance in the vicinity of the partition by locally reducing deflection and initial velocity notably.

In a study on the dynamic behaviour of floors supported on four columns and constructed with steel joists and concrete deck, Alvis (2001) applied different retro-fits to the original system. Vibration responses were measured and the floor performance ranked from 1 to 9. The retro-fits identified to contribute most to the improved performance were two posts located at the third points of mid-span of the floors. The floor performance was even more enhanced if spreader beams were fixed on top of the posts. Alvis proposed to improve floor performances by finding out the most annoying mode shape and placing retro-fits at the appropriate locations, whereas the best location would be an anti-node of that mode.

Khokhar (2004) and Khokhar et al. (2004) studied the effect of lateral element stiffness on dynamic performance of timber floors. The reference floor of 4.2 m × 3.66 m was constructed with LVL joists, spaced at 610 mm, decked with OSB panels and supported along all four sides. LVL blocking elements were then introduced at mid-span and connected to the floor joists by aluminium angles and different screw patterns, including different number of screws and in one case the additional use of adhesive. For further tests the LVL blocking elements and aluminium angels were replaced by solid lumber blocking, cross-bridging or cross-bridging plus strapping, with nails as connectors.

Using LVL blocking reduced the deflection in floor centre under 1 kN point load by 10% for the least stiff system and by more than 30% for the stiffest one. The fundamental frequency was lowered whereas higher frequencies were all raised, generally considerably. Testing the floor without the blocking but with the aluminium angles attached resulted in the largest reduction of the fundamental frequency and also in reductions of the higher frequencies. This was due to the mass of the aluminium angles. Therefore, comparison between the floors with LVL blocking and the one with only the aluminium angles attached would show raised fundamental frequencies for the use of blocking. Regarding the other lateral stiffening methods, cross-bridging plus strapping was the most effective way to lower the deflection, reducing it by about 25%. This also increased the fundamental frequency to a certain degree and significantly increased the higher frequencies. The other adopted methods little affected the fundamental frequency but increased the frequencies of higher modes. In general an increase in the stiffness of the lateral elements was found to lower the r.m.s. acceleration at various locations on the floors by up to 40%.

Bernard (2008) investigated the effect of various structural modifications on the rigidity and dynamic response of timber flooring systems constructed with I-joists or laminated glued lumber (LGL) joists. The floors had dimensions of 2.4 m × 4.8 m, were simply supported along the joist ends and decked with tongue and groove particleboard. Holes of different diameters drilled through the web of each I-joist at mid-span or quarter-span were found to hardly influence static rigidity and dynamic response. Floors decked with either 19 mm thick particleboard or 12 mm thick plywood sheets both exhibited similar dynamic performances and static rigidities. Varying the thickness of the plywood sheets from 9 mm to 12 mm and then to 19 mm was found to result in decreased resonances. Pre-compressing the particleboard sheeting in plane increased the rigidity by about 5% on average, but the influence on the dynamic performance was insignificant.

Introducing pine battens or steel furring channels into a flooring structure were reported to hardly influence the dynamic performance and to only affect higher modes.

The addition of a gypsum ceiling reduced fundamental frequencies and hardly affected the rigidity. Replacement of the gypsum by plywood little influenced the examined properties. Flooring systems with glued gypsum ceiling had little effect on lower frequencies but raised higher ones. Rigidity was increased by 4%, but this benefit was found to be offset by the increase in mass. Fixing the particleboard decking by adhesives in addition to nails showed an increase in rigidity by double percentage figures. The use of screws was similarly effective as the combined use of nails and adhesives. The effect on vibration modes was small. Using blocking was found to "*have an effect on dynamic behaviour in the lateral direction especially if glued and strapped*". Post-tensioned blocking that is accurately cut could "*eliminate higher lateral modes*". Some floors were used to examine the influence of post-tensioning in the longitudinal direction, and the tests showed no obvious effect on the dynamic performance. The flexural rigidity increased with a reduction in joist spacing. The effect of rubber inserts was depending on the location they were applied to. Damping was considerably raised when the rubber was applied at the locations where high shear stresses occur. Using rubber strips at midplane of I-joists largely reduced natural frequencies but highly increased damping.

3.3 Design criteria

Ohlsson (1982) proposed design criteria to rate floor vibrations and human discomfort. However, he stated that the presented criteria were not a straightforward outcome of the investigations carried out but based on those from other researchers' results and his own studies and experiences. The criteria, in particular the limiting values, were rather tentative.

The design criteria were first established with respect to heavy floors but considered to be applicable to light-weight floors with slight modifications. For the heavy floors, the weighted maximum compliance (dynamic flexibility or displacement/force) should be limited for continuous footfall loading and a fictitious initial velocity should be limited for transient footfall pulses. For light-weight floors the approach could be similar, but an additional mass would need to be coupled to the system to account for a person on the structure.

Ohlsson proposed to assume a damping ratio of 0.75% for light-weight floors since damping would decrease with increasing mass density. The damping ratio ζ is defined as the ratio of the damping coefficient c to the critical damping c_{cr} for a system, $\zeta = c/c_{\mathrm{cr}}$, with c_{cr} being proportional to $\sqrt{k \cdot m}$, so ζ would be consequently reduced as a result of added dead weight. The extensive research by Ohlsson (1982) was refined, resulting in a Swedish design guide on floor vibrations (Ohlsson 1988), in which a damping ratio of 1% was recommended for the design of conventional light-weight floors. The damping ratio could be reduced to 0.8% for floors of large span or large weight (> 150 kg/m^2).

The design guide (Ohlsson 1988) is applicable to floors with natural frequencies above 8 Hz only. The vibration response would then be controlled by limiting the static deflection and velocity responses. The reason for this can be explained from Figure 3.6.

According to Ohlsson’s work, a person walking produces high force components in the low-frequency range (Figure 3.6(a)). Variation of mobility (velocity/force) with frequency at a certain point on a floor is illustrated in Figure 3.6(b), which shows low mobility in the low-frequency range and high mobility around the resonances. The vibration velocity is then obtained from the product of the force and mobility (Figure 3.6(c)). Thus, "*the resultant vibration is made up of a low-frequency semi-static component [...] and a number of resonance dominated components [...], which are of the same magnitude [...], or larger than, the semi-static component*" (Ohlsson 1988).

Therefore, limiting the point load deflection of the floor would be needed to satisfy the floor performance for the semi-static component. The resonance dominated components should be controlled by limiting velocity responses as velocity was found to be the parameter most closely related to human discomfort from vibration with natural frequencies above 8 Hz (see also ISO 2631-2:1989).

The equation for calculating natural frequencies $\omega_{\mathrm{m,n}}$ of rectangular orthotropic plates simply supported along all four edges is as follows (Leissa 1969):

$$\omega_{m,n} = \frac{\pi^2}{L^2\sqrt{\rho}}\sqrt{D_{\mathrm{x}} m^4 + 2D_{\mathrm{xy}} m^2 n^2 \left(\frac{L}{B}\right)^2 + D_{\mathrm{y}} n^4 \left(\frac{L}{B}\right)^4} \quad \text{[rad/s]} \quad (3.3)$$

which can be modified with respect to $f_{\mathrm{m,n}}$ to:

$$f_{m,n} = \frac{\pi}{2L^2\sqrt{\rho}}\sqrt{D_{\mathrm{x}}m^4 + 2D_{\mathrm{xy}}m^2n^2\left(\frac{L}{B}\right)^2 + D_{\mathrm{y}}n^4\left(\frac{L}{B}\right)^4} \quad \text{[Hz]} \qquad (3.4)$$

where m and n correspond to the normal mode number in the x- and y-directions respectively, ρ is the mass density per unit area, L is the plate length, B is the plate width and D is the flexural plate stiffness where $D_{\mathrm{x}} > D_{\mathrm{y}}$ (for orthotropic plates) and:

$$D_{\mathrm{x}} = \frac{E_{\mathrm{x}}t^3}{12\left(1-\nu_{\mathrm{x}}\nu_{\mathrm{y}}\right)}; \quad D_{\mathrm{y}} = \frac{E_{\mathrm{y}}t^3}{12\left(1-\nu_{\mathrm{x}}\nu_{\mathrm{y}}\right)} \quad \text{and} \quad D_{\mathrm{xy}} = D_{\mathrm{x}}\nu_{\mathrm{y}}\frac{Gt^3}{6} \qquad (3.5)$$

where E is the modulus of elasticity (MoE), G is the rigidity modulus for shear and ν is the Poisson's ratio.

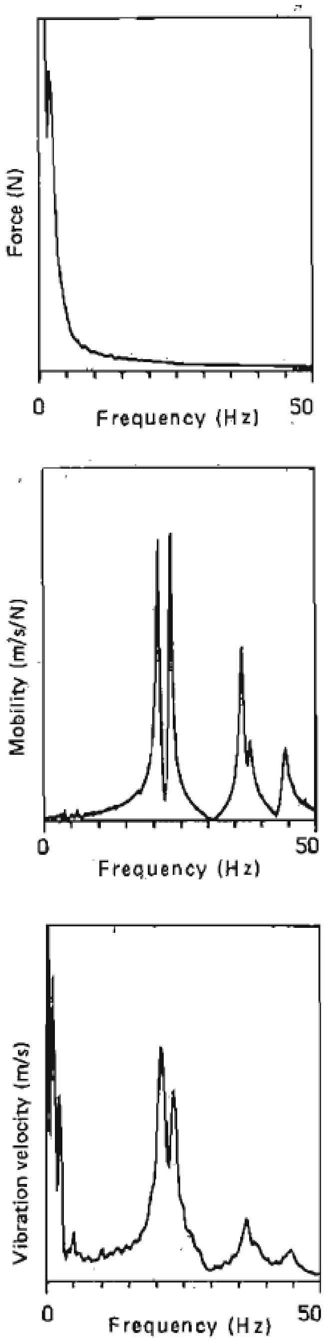
Eq. (3.4) was simplified by Ohlsson (1988) for calculating natural frequencies of first order modes, by assuming D_{xy} approximately equal to D_{y}, to:

$$f_{1,\mathrm{n}} = f_{\mathrm{n}} = \frac{\pi}{2}\sqrt{\frac{D_{\mathrm{x}}}{\rho L^4}}\sqrt{1+\left[2n^2\left(\frac{L}{B}\right)^2 + n^4\left(\frac{L}{B}\right)^4\right]\left(\frac{D_{\mathrm{y}}}{D_{\mathrm{x}}}\right)} \quad \text{[Hz]} \qquad (3.6)$$

Ohlsson plotted the values of the second square root term as a function of the modal number n against the standardised resonance frequency $f_{1,\mathrm{n}}/[(\pi/2)\sqrt{D_{\mathrm{x}}/\rho L^4}\,]$ for varied values of $D_{\mathrm{y}}/D_{\mathrm{x}}$ and L/B to serve as design charts (see Figure 3.7). For very low ratios of $D_{\mathrm{y}}/D_{\mathrm{x}}$ the term $(\pi/2)\sqrt{D_{\mathrm{x}}/\rho L^4}$ in front of the second square root term of Eq. (3.6) is approximately equal to the fundamental frequency and the second square root term becomes influential particularly for higher frequencies. The simplified method to calculate the fundamental natural frequency was thus given for low values of $D_{\mathrm{y}}/D_{\mathrm{x}}$ (≤ 0.01) as:

$$f_{1,1} = \frac{\pi}{2}\sqrt{\frac{D_{\mathrm{x}}}{\rho L^4}} \quad \text{[Hz]} \qquad (3.7)$$

Figure 3.7 and Eq. (3.6) show that if the stiffness ratio $D_{\mathrm{y}}/D_{\mathrm{x}}$ (the degree of isotropy) or the aspect ratio L/B is lowered, the number of first order modes within the frequency spectrum of interest is raised, and hence adjacent natural frequencies can become closer.



(a): Footstep force versus frequency caused by a person walking

(b): Mobility versus frequency, the peaks mark the resonance frequencies

(c): Vibration velocity versus frequency

Figure 3.6: Frequency spectra of footstep force, mobility and vibration velocity (Ohlsson 1988)

With regard to static loading, the most flexible point in a structure would be most relevant. For a typical timber floor, this would be at mid-span of the floor centrally between two joists since persons would not pay particular attention to joist positions.

For impulsive load the peak velocity response is to be determined. In case of continuous load, but only for floors with span above 4 m, or for floors with span equal to or smaller than 4 m but with intensive pedestrian traffic or large unobstructed areas, the r.m.s velocity needs to be calculated and classified as acceptable.

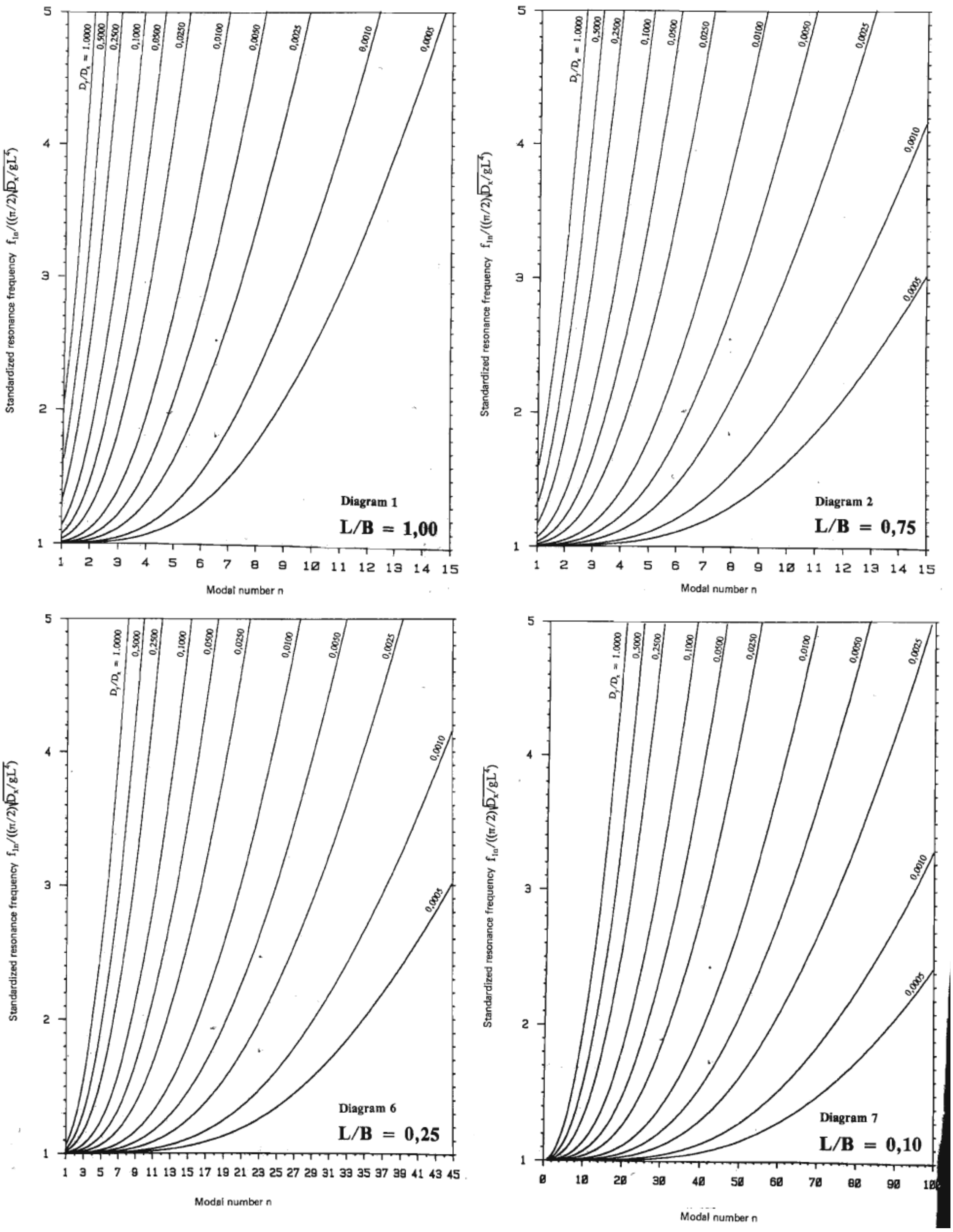


Figure 3.7: Typical design charts given by Ohlsson (1988)

The initial vertical vibration velocity, which is caused by an idealised vertical force impulse, is to be determined at the weakest point of the structure, the location with the maximum impulse velocity response. The simplified method for calculating the impulse velocity response $h'_{\max}$ of simply supported plates was proposed by Ohlsson as:

$$h'_{\max} = \frac{4(0.4 + 0.6n_{40})}{\rho BL + 200} \quad \text{[m/s/(Ns)] or [m/(Ns}^2\text{)]} \tag{3.8}$$

where n_{40} is the number of first order modes below 40 Hz, which can be determined from the design charts (see Figure 3.7).

Humans would be less sensitive to initial vibration velocity if the vibration is damped within a very short duration. Thus the unit impulse velocity response is limited with respect to a damping coefficient σ_0, which describes how quickly the vibration is damped:

$$\sigma_0 = f_{1,1} \cdot \zeta \quad \text{[Hz]} \tag{3.9}$$

It would be more complex if vibration possesses several frequency components, but Eq. (3.9) was regarded as possible suitable approximation. A preliminary proposal was given for classifying the impulse velocity response (see Figure 3.8).

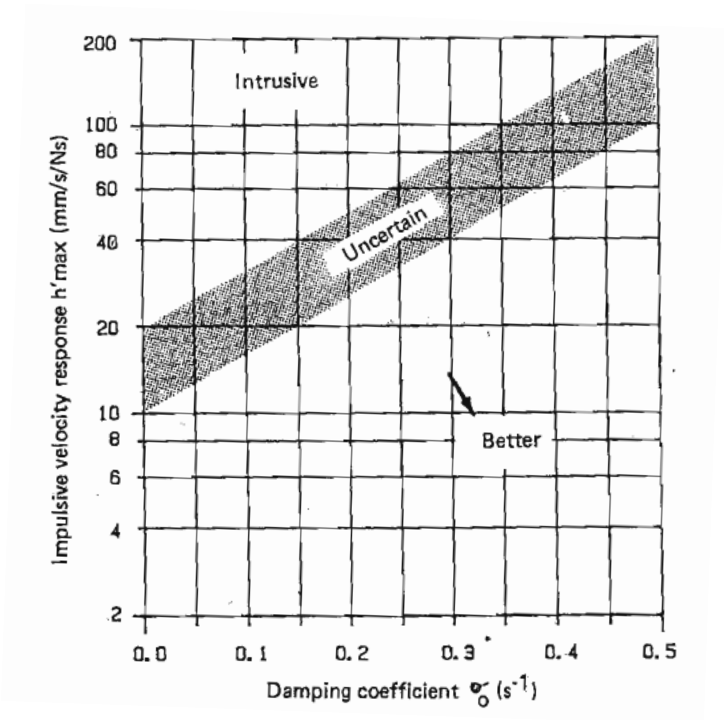


Figure 3.8: Preliminary proposal for classifying the response of a floor construction to an impact load (Ohlsson 1988)

Like the impulse velocity response, the r.m.s. velocity only includes the contribution of modes up to 40 Hz. The given equation for calculation contains some simplifications and is valid for simply supported rectangular plates:

$$w'_{\mathrm{rms}} = \frac{100}{\rho B L \sqrt{\zeta_{1,1}}} \sqrt{\frac{n_{1.2}^{\,2} + 1}{2 f_{1,1}^{\,3}}} \quad [(\mathrm{m/s})_{\mathrm{rms}}] \tag{3.10}$$

where $n_{1.2}$ is the number of first order modes with natural frequencies below $1.2 \times$ fundamental frequency and is also determined from the design charts (see Figure 3.7). To classify the r.m.s. velocity response, satisfactory values on comparable structures should be used. For the design regarding impulse and continuous load, the relative damping is assumed the same for all principal first order modes.

The methods presented by Ohlsson (1982, 1988, 1994) served as the basis for the vibrational serviceability criteria in the Eurocode 5 (EC5-1-1) (BS EN 1995-1-1:2004) (see also Ohlsson 1991, 1995). The Eurocodes had been established to serve as pan-European standards in form of harmonised design criteria within the member countries to build a common basis for design, research and development. The design of timber structures is covered in EC5-1-1. The criteria associated with timber floor vibrations are part of the serviceability limit states (SLS) in EC5-1-1. National Annexes (NAs) to EC5-1-1 provide modified or additional design criteria by considering local design situations. Referring to the EC5-1-1 it is thus required that for timber floors with a fundamental natural frequency greater than 8 Hz, the deflection under a unit point load and the unit impulse velocity response are limited. Equations for calculating the fundamental natural frequency, unit impulse velocity response and the velocity limit are provided in the code. A limit for the unit point load deflection and a method for calculating the deflection are not given. There is no guidance in the EC5-1-1 for floors with natural frequencies below 8 Hz.

Since in some countries the criteria for floor vibrations to EC5-1-1 are revised or superseded in the NAs, this issue was further investigated within a Short Term Scientific Mission (STSM) of COST Action E55 as part of the European Framework Programme 7 (FP7) in October 2007 (Weckendorf 2007), in which the design criteria of the different countries were compared and the implications of their use for floor construction of either country investigated. The detailed criteria of EC5-1-1 and NAs of the United Kingdom and Finland respectively are presented later in this section.

Chui (1987) and Smith and Chui (1988a) proposed designer usable methods for predicting the dynamic response of domestic timber floors, which are light-weight and rectangular plan, and have no openings and no intermediate support conditions. The floors consist of wood joists with semi-rigidly attached decking, and are simply supported along all edges. The methods included the determination of frequency-weighted r.m.s. acceleration to account for vibration amplitude, rate of decay and frequency components, and of natural frequencies. The equation for calculating the fundamental natural frequency was given as:

$$f_{1,1} = \frac{\pi}{2L^2}\sqrt{\frac{E_\mathrm{J} I_\mathrm{J}\left(n_\mathrm{J}-1\right)}{\rho_\mathrm{d} tB + \rho_\mathrm{J} bh\left(n_\mathrm{J}-1\right)}} \quad \text{[Hz]} \tag{3.11}$$

where E_J is the MoE of the joist, I_J is the second moment of area of the joist, h is the joist depth, b is the joist width, ρ_J is the density of the joist, n_J is the number of joists, ρ_d is the density of decking, t is the decking thickness. When the weight of a person, W_0 in [kg], at floor centre is accounted for, the equation becomes:

$$f_{1,1} = \frac{\pi}{2L^2}\sqrt{\frac{E_\mathrm{J} I_\mathrm{J}\left(n_\mathrm{J}-1\right)}{\rho_\mathrm{d} tB + \rho_\mathrm{J} bh\left(n_\mathrm{J}-1\right) + 4W_0 / L}} \quad \text{[Hz]} \tag{3.12}$$

It is required that the fundamental natural frequency is greater than 8 Hz as humans would be sensitive to the frequency range of 4 to 8 Hz.

The determination of r.m.s. acceleration is based on a single-degree-of-freedom analogue and thus concentrates on the fundamental vibration mode. The impact design amplitude accounts for the weight of the person performing the heel-drop test. The equation also includes some frequency-weighting factor, which is applicable to the frequency range of 8 to 80 Hz in accordance with ISO 2631. This finally resulted in the following expression for the r.m.s. acceleration $a_\mathrm{w,rms}$:

$$a_\mathrm{w,rms} = \frac{2000K}{m\pi\, f_{1,1}^{\;2}} \quad [\mathrm{m/s^2}] \tag{3.13}$$

where K is a factor determined from the fundamental natural frequency, corresponding damping ratio, and impact duration. The damping ratio is assumed to be 2% for unoccupied floors and 3% for floors when humans are present. It was found that the suitable limit for design would be $a_\mathrm{w,rms} < 0.45$ m/s^2 (see Section 3.1). For exact details see the publications by Chui (1987) or Smith and Chui (1988a).

In Canada studies were undertaken to develop a design criterion to limit floor vibrations based on consumer response (Onysko 1988). The existing criterion of limiting the deflection under uniformly distributed load was found to be inadequate with the increasing use of new construction techniques and materials and the changes in the material property assessment. Extensive field investigations by interviews and inspections for more than 600 floors of more than 100 buildings in different cities of Canada were conducted. The performance was classified into one of five categories if motion was noticed: *definitely acceptable*, *acceptable*, *undecided*, *not acceptable* or *definitely not acceptable*. Otherwise the floors were classified into the category *motion not noticed*. In the end, the two categories of *acceptable* and *definitely acceptable* and the two of *not acceptable* and *definitely not acceptable* were merged. Discriminant analysis was the method used to develop performance criteria. Peak dynamic response to an impulse loading, the damping ratio, the natural frequency, the deflection under concentrated load and the floor span were identified as the most important variables. Deflection under uniform load was found to be a poor discriminator for classifying floors. Therefore, the criterion could be based on the response to impulsive loading or on the response under static point load. The latter prevailed as for the former more reliable information on damping and assumed imposed mass on floors, such as furniture, would be required as they all influence the dynamic response. The suggested criterion, which yielded the lowest number of misclassified floors, is given as follows:

$$w_{\mathrm{d}} = \frac{7.217}{L^{1.274}} \le 1.75 \text{ [mm]} \tag{3.14}$$

with w_{d} as the limit for the deflection under 1 kN point load applied at mid-span of the floor, and L as the floor span in [m]. The criterion was then further calibrated and simplified as:

$$w_{\mathrm{d}} = \frac{8}{L^{1.3}} \le 2.0 \text{ [mm]} \tag{3.15}$$

The Canadian Wood Council (CWC) produced span tables based on this equation of deflection under a concentrated load and on the existing criterion of deflection under a uniformly distributed load, in which the more unfavourable one was chosen. As described by Onysko et al. (2000), this method was finally adopted in Part 9 of the National Building Code of Canada (NBCC) in 1990. As this criterion was particularly aimed at solid sawn lumber floors, a modified method was developed for the Canadian Construction Materials Centre (CCMC) for applying this method to floors constructed with engineered wood components. This limited the existing criterion to floor spans up to 5.5 m.

For spans between 5.5 m and 9.9 m, the limit for the deflection is calculated as:

$$w_{\mathrm{d}} = \frac{2.55}{L^{0.63}} \quad \text{[mm]} \tag{3.16}$$

For spans above 9.9 m the limit of point load deflection was set to 0.6 mm.

Dolan et al. (1999) proposed a design criterion solely based on the fundamental natural frequency. The criterion was validated with laboratory tests on double T-beam specimens and floor specimens and tests on occupied and unoccupied floors in buildings. The floor span varied between 2.7 m and 8.5 m. The decking consisted of single layer of oriented strand board (OSB) or plywood or of double layers of an underlayment panel product and plywood, and was attached using glue and nails. Researchers judged the floor response and classified it as *acceptable*, *marginally acceptable* or *unacceptable*. The excitation was induced by a heel-drop from a researcher. The proposed criterion is that the fundamental frequency of unoccupied floors is greater than 15 Hz and of occupied floors greater than 14 Hz. Composite action between subflooring and joists should be neglected in the calculation of the fundamental natural frequency.

New tentative design criteria were developed at Forintek Canada Corp as reported by Hu (2000, 2002), followed by accordingly enhanced calculation procedures from Chui (2002) and the design criteria and calculation procedures modified and presented by Hu and Chui (2004a, 2004b), with the intention to cover the vibration design of a broad range of wood-based floors.

In the study by Hu (2000, 2002), the five presented criteria were based on the test results of 112 Canadian field floors. Nearly half of those floors were constructed with engineered timber joists and the rest with lumber joists. Bridging between joists was used for half of the floors, and one quarter of the floors had a micro concrete layer on top of a wood-based sub-floor. Clear floor spans usually ranged from 3 m to 8 m, for two floors from 11 m to 12 m. The floors were tested with respect to 1 kN static deflection, fundamental frequency and corresponding modal damping ratio, 1 Ns impulse peak and r.m.s. acceleration and 1 Ns impulse initial velocity. All floors were subjectively rated by occupants or researchers. Approximately 63% of the floors were rated acceptable. To determine the fundamental frequencies, damping ratios and mode shapes, tests with two different types of excitation were conducted on each floor. In one case, a shaker induced random vibration, while in the other case, a person on the floor performed impact tests, stimulating the impacts by dropping a ball or striking an instrumented hammer. The floor was excited offset from mid-span on the joist adjacent to the central one. Responses at the mid-span of at least six floor joists were recorded. To detect acceleration responses, an impact force, which lasted for about 30 ms, was generated by dropping a ball at the mid-span of the central joist, and the response was measured at the same location. The signal was integrated to yield velocity. Both acceleration and velocity were then normalised under consideration of the impact force to obtain 1 Ns impulse responses. Static deflection was induced by the weight of a person at different locations along the central lines of each joist, and dial gauges were used to measure the deflection.

Four existing design criteria were reviewed first and information from the database was used to investigate three of them. The impulse velocity criterion established by Ohlsson (1982) and adopted in EC5-1-1 was questioned but the appropriate unit impulse velocity responses tried to be obtained experimentally. Using theses responses to assess the vibrational floor performances classified all floors as acceptable, which led to a failure of the comparison. It was assumed that the 1 Ns impulse initial velocity used was different from the one adopted in EC5-1-1. Thus, there would be still a need to investigate how this unit impulse velocity response can be measured. The aforementioned design method by Dolan et al. (1999) was found to be working well for light-weight floors but too conservative for heavy floors.

The database was then used to develop new serviceability criteria. Logistic regression was applied to perform correlation studies between human perception of floor vibrations and floor performance variables. The selected variables for this study included clear span, mass per unit area, 1 kN static deflection, fundamental frequency, 1 Ns impulse initial velocity, peak and r.m.s. accelerations, but not damping. In the logistic regression on different combinations of performance variables, a number of serviceability criteria were developed. The five criteria with highest accuracy were detailed as follows with 101 of the 112 floors included in each analysis:

(1) 1 kN concentrated load deflection and fundamental natural frequency;
(2) 1 kN concentrated load deflection, mass per unit area and fundamental natural frequency;
(3) Clear span, fundamental natural frequency and 1 Ns impulse initial velocity;
(4) Clear span, fundamental natural frequency and 1 Ns impulse r.m.s. acceleration;
(5) Clear span, fundamental natural frequency and 1 Ns impulse peak-acceleration.

Based on the analysis, the following criteria were established, whereas for the first two, stricter alternatives were also provided:

(1a) Frequency [Hz] / (1 kN deflection [mm])$^{0.39}$ ≥ 15.3;
(1b) Frequency [Hz] / (1 kN deflection [mm])$^{0.39}$ ≥ 16.2
(2a) Frequency [Hz] × (mass/area [kg/m²])$^{0.265}$ / (1 kN deflection [mm])$^{0.219}$ ≥ 37.1;
(2b) Frequency [Hz] × (mass/area [kg/m²])$^{0.265}$ / (1 kN deflection [mm])$^{0.219}$ ≥ 41.5;
(3) Frequency [Hz] × (span [m])$^{0.35}$ / (1 Ns impulse initial velocity [mm/s])$^{0.21}$ ≥ 17.5;
(4) Frequency [Hz] × (span [m])$^{0.38}$ / (1 Ns impulse r.m.s. acc. [mm/s²])$^{0.21}$ ≥ 10.3;
(5) Frequency [Hz] × (span [m])$^{0.49}$ / (1 Ns impulse peak-acc. [mm/s²])$^{0.143}$ ≥ 10.7.

These criteria yielded the following accuracy for correctly classifying the floors:

(1a) 86.1%;
(2a) 87.1%;
(3) 91.1%;
(4) 90.1%;
(5) 90.1%.

It was found that all of the above mentioned criteria largely enhanced the classification of floors compared to existing criteria. Although the first method performed worse than the others, it was regarded as the best choice due to simpler and more reliable calculation and measurement procedures than those for e.g. velocity and acceleration. The criterion was further validated using a second database with 58 floors tested in field or laboratory.

Following, inter alia, this research by Hu (2000, 2002), Chui (2002) proposed enhanced calculation methods based on the ribbed-plate theory to predict the deflection under a point load and the fundamental natural frequency. He believed current methods for determining the fundamental frequency not always sufficiently to account for details of the flooring system, in particular for the two-way action of certain timber floors. The proposed equation for estimating the fundamental natural frequency is basically the one shown in Eq. (3.4), simplified for the fundamental mode of vibration to:

$$f_{1,1} = \frac{\pi}{2\sqrt{\rho}} \sqrt{D_{\mathrm{x}} \left(\frac{1}{L}\right)^4 + 2D_{\mathrm{xy}} \left(\frac{1}{LB}\right)^2 + D_{\mathrm{y}} \left(\frac{1}{B}\right)^4} \quad \text{[Hz]} \tag{3.17}$$

Chui then gave further methods for calculating the properties of floor components such as composite joist flexural rigidity, bridging system flexural rigidity and joist torsional constant. The models were verified using the results form 29 test floors, including floors tested by Hu (2000). He finally found that the deflection under 1 kN point load was well predicted but the fundamental frequencies were mainly overestimated. The models were considered to be acceptable for design purposes.

The investigation conducted by Hu (2000, 2002) and Chui (2002) was further adjusted and harmonised (Hu and Chui 2004a, 2004b). The regression analysis was extended to 106 field floors, resulting in the flowing criterion:

- Fundamental frequency [Hz]/(1 kN deflection [mm])$^{0.44}$ $\geq$ 18.7.

The presented design method was considered to provide a framework for formulating an acceptable design method for a wide range of timber floors. It was assumed that further calibration of the criterion might be needed so as to be applicable to floor designs in other countries since it was developed with focus on Canadian material properties.

Toratti and Talja (2006) suggested that floors should first be classified as low- or high-frequency floors depending on a threshold level of 10 Hz of the fundamental frequency, and then different controlling criteria should apply. Furthermore, a classification of residential and office buildings into five different categories was proposed, with the intention to provide guidance for costumers and contractors. The categories are labelled from A to E with C as the base class, A and B as higher classes and D and E as lower classes. The classification table presented by Toratti and Talja (2006) is reproduced in Table 3.3. These classes consider the vibration perception of a seated person and the vibration of objects. The vibration is assumed to be induced by walking. The proposed design limits for low- and high-frequency floors are summarised in another table, which is reproduced in Table 3.4.

A factor, being effective if neither the span nor the width of the floor exceed 6.0 m, was suggested to apply onto the limits illustrated in Table 3.4 since it was found that dynamic forces due to walking would be lower in smaller rooms. The factor can be determined from a chart or by calculations. More details can be found from the publication by Toratti and Talja (2006). The limiting values were verified against the data collected from experimental investigations on steel, timber and concrete floors over a period of ten years. The vibration was usually induced by a person walking over the floor and rated by 3 to 15 seated persons individually whereas the acceptability classification was dependent on the opinion of the majority. The rating was based on body sensing and on the visual or aural effects of vibrating objects, such as coffee cup with saucer and spoon, leafs of a plant or water in a glass, which were placed on a tripod.

Table 3.3: Vibration classes proposed by Toratti and Talja (2006)

A	Special class for vibrations inside one apartment. Normal class for vibrations transferred from another apartment. The vibration is usually imperceptible.
B	Higher class for vibrations inside one apartment. Lower class for vibrations transferred from another apartment. The vibration may be perceptible but usually it is not annoying (inside one apartment).
C (base class)	Normal class for vibrations inside one apartment. The vibration is often perceptible and some people may feel it annoying (inside one apartment).
D	Lower class for vibrations inside one apartment, e.g. attics and holiday cottages. The vibration is perceptible and most people feel it annoying (inside one apartment).
E	Class without restrictions.

Table 3.4: Design limits for vibration classes (Toratti and Talja 2006)

	Dynamic vibration values				Static deflection values	
	$f_{1,1} < 10$ Hz	$f_{1,1} > 10$ Hz				Floor plate or super-structure
	$a_{w,rms}$ [m/s^2]	v_{max} [mm/s]	v_{rms} [mm/s]	u_{max} [mm]	δ [mm/kN]	δ_1 [mm/kN]
A	≤ 0.03	≤ 4	≤ 0.3	≤ 0.05	≤ 0.12	≤ 0.12
B	≤ 0.05	≤ 6	≤ 0.6	≤ 0.1	≤ 0.25	≤ 0.25
C	≤ 0.075	≤ 8	≤ 1.0	≤ 0.2	≤ 0.5	≤ 0.5
D	≤ 0.12	≤ 10	≤ 1.5	≤ 0.4	≤ 1.0	≤ 1.0
E	> 0.12	> 10	> 1.5	> 0.4	> 1.0	> 1.0

Note: $a_{w,rms}$ is the weighted r.m.s. acceleration, v_{max} is the peak velocity, v_{rms} is the r.m.s velocity, u_{max} is the peak vertical displacement, δ is the global deflection, and δ_1 is the local deflection.

Analysing the results from the vibration measurements showed that the worst distinction between acceptable and unacceptable floors was determined by dynamic displacement.

Frequency-weighted r.m.s. acceleration gave the best correlation with subjective ratings of low- and high-frequency floors. Peak velocity was found to be a similar good indicator for the latter. However, vibrational performances of high-frequency floors were proposed to be controlled by static deflection as it would classify the floors reasonably well and be simpler handled in design. Another aspect to be considered would be the local deflection, being determined at a location with 600 mm distance to the applied load. Both global and local deflections need to be limited. Low-frequency floors were proposed to be controlled by limiting the acceleration response. Class C was used as a demarcation separating acceptable floors from unacceptable ones.

Two design equations for the determination of the frequency threshold were given. One formula considers the stiffness in both directions for floors supported on all four edges and the other one includes only stiffness in the span direction. The equations are equivalent to those in the Finnish National Annex (see Table 3.6). A 30 kg/m^2 service load is to be added to the floor mass for the calculation.

For the design of high-frequency floors, there were also two formulae provided for determining the global deflection, one based on the equation for calculating the deflection of an orthotropic plate, simply supported along all edges, and the other one based on the equation for calculating the deflection of a single beam under point load, the higher calculated deflection being adopted for design. The local deflection was proposed to be obtained experimentally as it would often be hardly predictable by engineering calculations. The global and local deflections are both to be determined under a 1 kN point load. The design procedure regarding low-frequency floors was not further commented.

In the work by Weckendorf (2007), the design criteria with respect to EC5-1-1 used in Finland and the UK were compared as presented in detail subsequently. To design the floors regarding their dynamic performance, the design rules of EC5-1-1 are adopted in the UK. Guidance for determining the deflection and its limit is introduced in the UK National Annex to BS EN 1995-1-1:2004 (UK NA to EC5-1-1) due to a lack of formula and limiting value regarding the deflection criterion in EC5-1-1. The damping ratio for determining the design limit of the unit impulse velocity response in EC5-1-1 is doubled in the UK NA to EC5-1-1. The calculation methods for the velocity response and its limit are questionable since their validation is not easily proven (Hu 2000; Hu et al. 2001; Zhang 2004).

Completely new design criteria were established in the Finnish National Annex to EN 1995-1-1:2004 ((FI NA to EC5-1-1) (VTT 2007)). The floors are first classified as low- or high-frequency floors at a threshold of 9 Hz, and then only a deflection limit is applied for high-frequency floors. Guidance for low-frequency floors is not given. Table 3.5 illustrates a comparison of the design guidelines for timber floors used in the UK and Finland. Table 3.6 provides the calculation methods used and Table 3.7 shows the limiting values. For determination of the individual factors k, see the UK NA to EC5-1-1 and FI NA to EC5-1-1.

Table 3.5 : Floor classification and design guidance in the UK and Finland

Country	**Low-frequency floor**		**High-frequency floor**	
	Condition	Guidance	Condition	Guidance
UK (based on EC5-1-1)	$f_{1,1} \leq 8$ Hz	N/A	$f_{1,1} > 8$ Hz	1) Limiting unit point load deflection w 2) Limiting unit impulse velocity response h'_{max}
FI (NA)	$f_{1,1} < 9$ Hz	N/A	$f_{1,1} \geq 9$ Hz	Limiting unit point load deflection δ

As can be seen from the design rules, classification of the structures as low- and high-frequency floors and assessment of their dynamic performances differ in the UK and Finland. In the FI NA to EC5-1-1 there are two formulae provided to calculate the fundamental frequency. One is used for floors supported along two edges and the other one for floors supported along four edges. The formula used for floors with free outer edges is the one used in EC5-1-1 "*for a rectangular floor [...] simply supported along all four edges [...].*" This comment in the EC5-1-1 is unhelpful as it gives the impression that the formula is rather to be used for floors supported along four edges. However, this simplified formula may be more accurate for two-side supported floors but is also used for floors supported along four sides. The Finnish formula for floors with supports along all sides considers stiffness in transverse direction, which is neglected in the simplified equation.

Table 3.6: Design equations for calculating frequency, deflection and velocity

Country	**Fundamental frequency** [Hz]	**Point load deflection** [mm]	**Velocity response** [m/s/(Ns)]
UK	"*For a rectangular floor [...], simply supported along all four edges [...]*" (EC5-1-1): $$f_{1,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{\rho}}$$	$$w = \frac{k_{\text{dist}} 1000 L_{\text{eq}}^{3} k_{\text{amp}}}{48(EI)_{\text{joist}}}$$	$$h'_{\max} = 4 \cdot \frac{(0.4 + 0.6\, n_{40})}{\rho L B + 200}$$
FI (NA)	For 2-side supported floors: $$f_{1,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{\rho}}$$ For 4-side supported floors: $$f_{1,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{\rho}} \cdot \sqrt{1 + \left[2\cdot\left(\frac{L}{B}\right)^2 + \left(\frac{L}{B}\right)^4\right]\cdot\frac{(EI)_{\text{B}}}{(EI)_{\text{L}}}}$$	$$\delta = \min\begin{cases} \dfrac{FL^2}{42 \cdot k_\delta \cdot (EI)_{\text{L}}} \\ \dfrac{FL^3}{48 \cdot s \cdot (EI)_{\text{L}}} \end{cases}$$	N/A

Note: $(EI)_{\text{L}}$ and $(EI)_{\text{B}}$ are the equivalent plate bending stiffness about an axis perpendicular and parallel to the beam direction respectively in [Nm^2/m], $(EI)_{\text{joist}}$ is the bending stiffness of a joist in [Nmm^2], L_{eq} is equal to L in [mm] multiplied by a factor depending on support condition, ρ is the mass density per unit area in [kg/m^2], and s is the joist spacing in [m].

It can be noted that the formulae to calculate the deflection are adopted from the general deflection equations for beams and plates but have been modified in the UK NA to EC5-1-1 to account for factors such as load distribution. Furthermore, detailed guidance on the design of more complex flooring structures is not given. The design criteria were used to assess the vibration performance of a simple and a complex structure from the UK and Finland respectively (Chapter 7).

Table 3.7: Design limits for frequency, deflection and velocity

Country	Fundamental frequency	Point load deflection	Velocity response
UK (NA)	$f_{1,1} > 8$ Hz	1.8 mm/kN for $L \leq 4000$ mm $16500/L^{1.1}$ mm/kN for $L > 4000$ mm	$h'_{\max} \leq b^{(f_1 \zeta - 1)}$ [m/s/(Ns)] where $\zeta = 0.02$ (EC5: $\zeta = 0.01$)
FI (NA)	$f_{1,1} \geq 9$ Hz	$0.5 \times \min \begin{cases} \sqrt[4]{\dfrac{(EI)_{\mathrm{B}}}{(EI)_{\mathrm{L}}}} \\ \dfrac{B}{L} \end{cases}$ mm/kN for $L \leq 6000$ mm 0.5 mm/kN for $L > 6000$ mm An additional 0.5 mm deflection can be allowed in case of floating and raised floors	N/A

Note: b is a constant, see EC5-1-1 or the UK NA to EC5-1-1 for determination.

3.4 Predicting dynamic floor response using numerical methods

To predict dynamic floor responses, Ohlsson (1982) used a grillage model in which each beam of a floor was represented by a beam member. For floors with the deck fixed by screws, the beam member would have the mass and flexural rigidity of the corresponding actual beam, whereas for the floors with glued deck, the flexural rigidity of a composite T-beam section should be assumed. Decking and ceiling boards would be represented by cross beams. It was intended to model one of the flooring systems tested in laboratory. As the floor had no ceiling attached and no glue was used, a torsion-weak model was assumed, which finally consisted of five main and five cross beams. The computer programme SFVIBAT-II was used for a dynamic analysis to obtain mode shapes and natural frequencies. An excellent correlation between measured and predicted mode shapes was observed, and the predicted natural frequencies satisfactorily matched the measured data. It was thought that a better correlation of predicted and measured frequencies may have been obtained by considering torsional stiffness and elasticity of the connections.

Chui (1987) developed a mathematical model based on the Rayleigh-Ritz method to predict dynamic responses of timber floor types shown in Figure 3.9. Every joist end was assumed to be simply supported, while the two outer joists could be simply supported or free along their length. The decking could be rigidly or semi-rigidly connected to the joists. The model was validated by comparing the responses obtained from experimental work to those predicted by the model. The results for comparison also included floor responses measured by Ohlsson (1982).

The mode shapes predicted were found to be identical to the ones determined experimentally. Also the measured and predicted fundamental frequencies correlated well with a variation of up to 5% in general and a maximum of 13%. The natural frequencies of higher vibration modes were generally underestimated by mostly below 20% with increasing inaccuracy for successive modes. This was thought to be caused by neglecting transverse shear deformations in the model since these deformations would get more significant with increasing mode number.

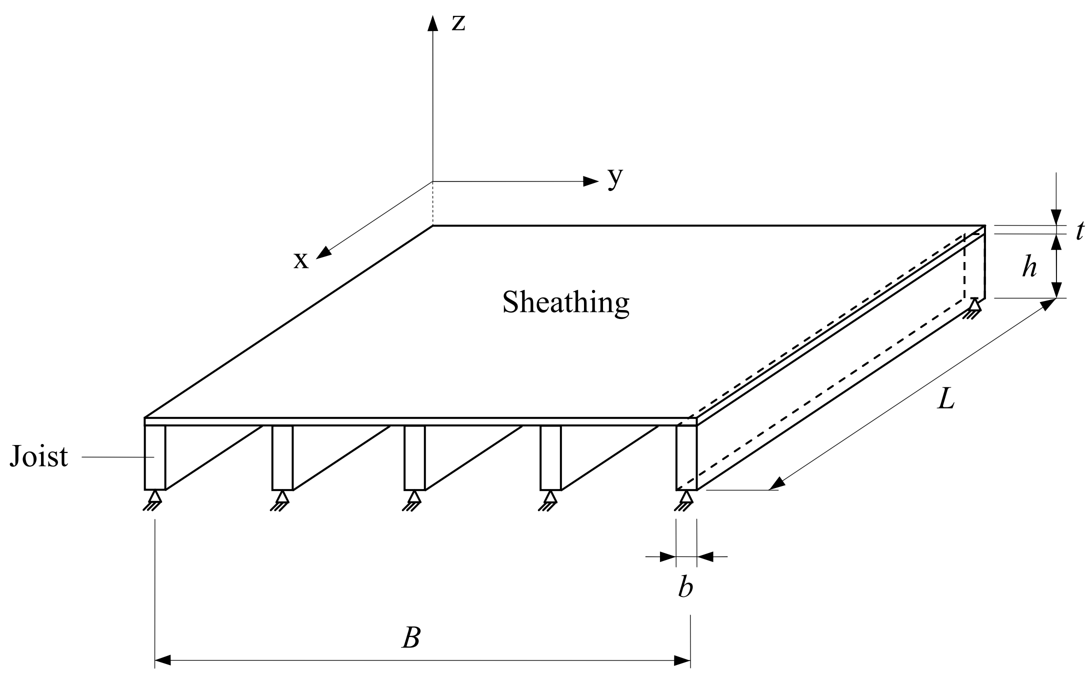


Figure 3.9: Type of floor considered in Chui's model (Chui 1987)

The predicted r.m.s. acceleration differed from measured values by 5% to 45% with an average of 25%, while there was the tendency of overestimating the responses. Possible reasons for this were found to be lower modal stiffness or mass assumed in the model than the actual values of the floor, or inappropriate damping used. Due to good prediction of fundamental frequencies, it was thought that lower assumed stiffness in the transverse direction or inappropriate damping were more likely to be the causes. The predicted vibration magnitudes would also be dependent on the accuracy of the predicted frequencies. It was concluded that the model is acceptable for the purpose of design, and was then used for sensitivity studies on floor performances with modifications in floor specifications (see also Section 3.2).

Hu (1992) developed a numerical model for predicting natural frequencies and acceleration responses of ribbed plates. The model was developed based on the modal synthesis method and experimentally validated by conducting vibration tests on timber I-joist floors (see Section 3.2). With respect to the prediction of natural frequencies of the floors, errors under 10% were observed for 29 floors and errors above 25% for three floors. The model was also validated against the test results of other 17 I-joist floors, which were not specifically tested for the described purpose. Similar agreement levels as before were generally found when predicting the higher frequencies, and an error of 7.4 % on average for the fundamental frequencies. Comparisons of mode shapes were undertaken for three test floors. The shapes and the number of nodes and anti-nodes were found to be predicted well and the magnitudes of the mode shapes estimated reasonably.

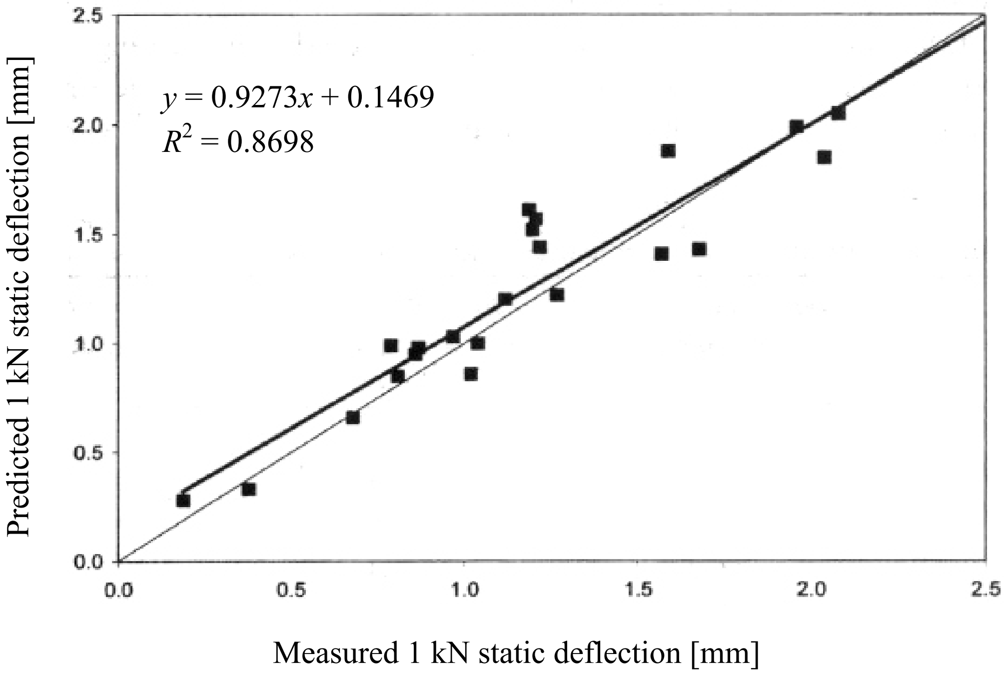
The model was identified to yield more accurate predictions of the natural frequencies for the I-joist floors investigated than the models developed by Chui (1987, see above) and Filiatrault et al. (1990). The higher accuracy was not fully given for predicting the natural frequencies of two floors with lumber joists, one tested by Chui (1987) and the other one by Ohlsson (1982), comparing Hu's model predictions to Chui's and Filiatrault's, and Chui's and Ohlsson's respectively. Hu’s model, which considers the effects of shear deformations and rotatory inertia in ribs, was concluded to be applicable for predicting natural frequencies and mode shapes of ribbed plates of various materials, due to no restrictions to material types, with a similar or better accuracy compared to other models.

In the studies by Hu et al. (2002), Jiang and Hu (2002) and Jiang et al. (2004), a finite element (FE) model was established for predicting fundamental frequencies and point load deflections of wood-based floors. Four-noded quadrilateral shell elements were used to model deck and ceiling, two-noded general beam elements to model joists and the transverse stiffening members. To model the fasteners, special connector elements were developed (see Jiang et al. 2004 for details). A two-noded connector element for semi-rigid connections of joists and transverse members and a four-noded interface element for modelling connection of deck or ceiling to joists were adopted.

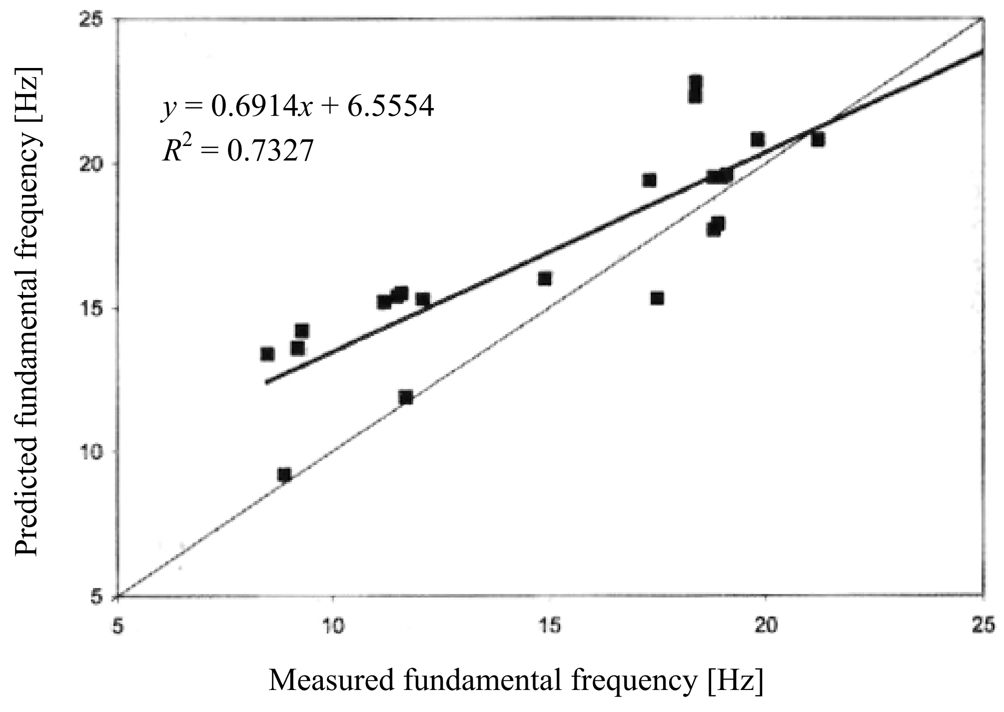
The withdrawal modulus of fastener-wood connections was experimentally determined using monotonic loading tests. Since this could only determine the modulus in one direction, the moduli of withdrawal and compression were assumed the same, otherwise more sophisticated techniques would be required to consider varied moduli. After performing sensitivity studies on the different moduli, a slip modulus of 1.5×10^6 N/m and an axial modulus of 3×10^8 N/m per fastener were found to be reasonable for predicting static deflections. For predicting dynamic response, the axial modulus was set to 3×10^5 N/m per fastener.

To verify the FE model, the predictions were compared to results obtained from measurements on 22 flooring systems constructed with I-joists or truss joists. Other construction parameters varied. Figure 3.10 shows such comparison of predicted deflections and fundamental frequencies with the measured ones. The deflections were found to be well predicted and the fundamental frequencies to be generally over-predicted. The over-prediction of the frequencies was thought to be caused by the support conditions modelled as simply rigid. Modelling one floor with flexible boundary conditions showed that the floor was sensitive to that change.

The accuracy of predicting relative changes in fundamental frequencies and static deflections due to structural modifications was also investigated by examining the variation in vibrational parameters of eight floors due to the introduction of transverse reinforcements. It was found that the prediction of relative changes worked rather well and that the contribution of the transverse reinforcements was slightly underestimated by the model. This led to conservative predictions, which was considered beneficial in the sense of avoiding excessive vibrations. The model was found reliable and unique particularly in the case of wood-based floors with transverse stiffening members.



(a) Correlation of measured and predicted deflections under 1 kN static loading



(b) Correlation of measured and predicted fundamental frequencies

Figure 3.10: Comparison of measured and predicted parameters (Jiang et al. 2004)

3.5 Summary and discussion

This chapter has reviewed the results of relevant previous research and the development of design standards and guides on the design and assessment of timber flooring systems with respect to dynamic response. The human sensitivity to vibrations, experimental investigations on dynamic floor response, proposals for controlling the vibrational behaviour and the predictability of response parameters using mathematical methods were all considered.

The previous research showed that damping is a very important factor influencing the sensitivity of humans to dynamic floor responses. However, some issues are not yet clarified, e.g. how damping should be considered in design, whether it should be used as a relative value such as damping ratio or as a coefficient depending on frequency, whether damping can be assumed to be the same for individual vibration modes, whether imposed load enhances or worsens damping capacity, etc. Humans on floors increase the damping ratio. If adding other types of loading, a negative effect on damping could sometimes be observed but also positive effects were reported. The determination of damping is complex, its accuracy also depending on test procedures and data analyses. The proposed damping ratios for serviceability design of timber flooring structures vary considerably (e.g. 0.8% to 3% for the assessment, 5% for the structural design). Inspection of damping ratios of not only the fundamental mode is of interest since some researchers highlight that higher frequency components contribute to annoyance from vibrations, sometimes significantly. Tests indicated that at least the first four modes are well-excited by walking or a heel-drop. The influencing degree of higher frequency components is also dependent on the spacing between adjacent natural frequencies. They can be more separated by increasing the stiffness in transverse direction as indicated in the equation for determining the natural frequencies of orthotropic plates supported along all edges and in the results on timber floors with varied configurations. The frequency range of interest was shown to be 0 Hz - 40/50 Hz.

In the previous research, the investigations of the effect of structural modifications on dynamic performance of timber floors were conducted experimentally, analytically and numerically with variation in types and locations of excitation and number and locations of measurement points. Response measurements in floor centre may normally be dominated by the response of the fundamental mode, but higher modes with nodes in the floor centre may not be observed at that location. As dynamic properties cannot always be determined accurately using simplified design equations, numerical models were developed to predict dynamic variables. Modal shapes of different vibration modes were normally predicted well with varied accuracies in the predicted natural frequencies.

Stiffness, mass and damping are the major properties in vibration response. Some design procedures only partially consider these properties. All criteria usually account for a stiffness effect but may ignore the effect of mass or damping. Determination of deflection normally accounts for contribution of stiffness, determination of fundamental frequency for mass and (mainly longitudinal) stiffness. The unit impulse velocity response accounts for the contribution of higher vibration modes to overall free vibration response. Its limiting value is controlled by damping together with the fundamental frequency. Threshold values of fundamental natural frequencies vary for different criteria, e.g. 8 Hz, 9 Hz, 10 Hz, 14 Hz, 15 Hz or even a value depending on static deflection. The lower of those threshold levels are aimed to distinguish between low- and high-frequency floors, whereas after classification, further controlling mechanisms are required. In the literature under review suggestions are given for the high-frequency floors but in majority not for low-frequency floors. Timber flooring systems are commonly light-weight and thus usually categorised as high-frequency floors.

The diversity of design criteria is a consequence of simplifying complex problems with a high number of influential construction properties and response variables. The latter are mainly difficult to predict and all of these parameters appear not all controllable at the same time. A lack of information about certain parameters also contributes to this, such as reliable damping ratios assignable to different types of common timber flooring structures. Some criteria were established with respect to local and state-of-the-art construction practices. Due to the advent of engineered structural timber elements, which are increasingly used, and varied construction practices in different countries, design procedures were modified accordingly without identifying a universally applicable design method, even not within Europe with respect to the EC.

The literature review, the various results and suggestions, show that more information on the dynamic response of timber flooring systems is required, so as to further the vibrational behaviour of the floors, to examine the current design criteria and to enhance the assessment. To obtain these aims, it is required to conduct significant experimental investigations as a basis for subsequent analytical studies. State-of-the-art construction techniques are to be applied to form the flooring structures, using engineered timber joists such as composite timber I-joists or metal-web joists. Using advanced modal analysis techniques can assist to more accurately identify the modal parameters of the structures. Since damping is recognised as highly influential on human sensitivity and largely influences the structural design against disturbing vibrations (e.g. EC5-1-1), the damping ratios obtained experimentally need to gain particular attention. Also studies on the EC5-1-1 design criteria are required as they are modified in member countries although the Eurocodes have been established to harmonise the design.

As further shown by the literature review, mathematical and numerical approaches have been conducted to predict the dynamic response of flooring systems. However, the information on computer-based FE models for timber flooring systems to predict their performances is very limited. Therefore, modelling timber I-joist flooring systems within this research may contribute to more reliable numerical eigenproblem analysis.

The following chapters introduce the experimental test series, the employed measuring and modal identification techniques, present the observation and discussion of the test results, the examination of EC5-1-1 design criteria and the detailed study of damping ratios, so as to finally arrive at recommendations for structural design and vibration assessment.

4. Experimental investigations and modal analysing methods

In laboratory conditions dynamic responses of sixty-seven timber floor configurations of varying structural and non-structural properties were experimentally investigated wherein thirty were built with TJI joists, twenty-eight with JJI joists and nine with metal-web joists. Initially, these three groups of floors are treated separately in the following subsections whereas thereafter similarities or clear differences in dynamic properties between these groups are identified and handled appropriately. The effect of the structural and non-structural floor modifications was examined. The exact details are given in the Sections 4.1 - 4.3 whereas the following overview summarises all investigated parameters:

- Floor decking types
- Fixing methods for decking to joists connection: screws versus adhesives + screws
- Support conditions
    - simply supported versus screw-fixing
    - two versus four sides supported
- Floor dimensions
- Joist depth
- Joist arrangements
    - joist spacing
    - double joists
- Imposed load
- Transverse stiffening
    - I-joist blocking: 0, 1 and 2 rows
    - Strongback
        - Type: Kerto S versus solid timber
        - Dimensions
        - 0, 1 and 2 rows
- Ceiling

All floors were tested with regard to static deflection and their modal properties: frequencies, damping and shapes. The unit impulse velocity response as suggested by Ohlsson (1982) and BS EN 1995-1-1:2004 (EC5-1-1) can hardly be measured directly and validated (Hu 2000; Hu et al. 2001). However, parameters used to calculate the velocity response and its limit as provided in EC5-1-1 and National Annex to BS EN 1995-1-1 2004 (UK NA to EC5-1-1) can be determined experimentally, including the damping ratio, the number of first order modes below 40 Hz and the fundamental frequency. Details of the test conduction are described in the Sections 4.4 and 4.5.

4.1 Structural detailing of TJI joist floors and support conditions

It was aimed at reaching conditions, which were as close to common floor construction practice as possible for this laboratory test series. The materials and test environment for the investigations of the TJI joist floors were provided by specialist timber frame supplier Oregon Timber Frame Ltd. The testing programme was prepared in cooperation with their specialists. Their expertise in floor construction was utilised for assembling the test floors and accomplishing the required modifications.

The materials for the test floors were the same as used in practice by Oregon Timber Frame Ltd. Alterations were made for the decking boards, boundary conditions and deck to joist fixing methods.

Full scale timber flooring systems with two different floor sizes and I-joist depths were constructed. For floors of both dimensions a test series including fifteen variations was established, resulting in a total of thirty floor configurations to be investigated. The two floors had dimensions of $L \times B = 3.70 \text{ m} \times 4.40 \text{ m}$ with 200 mm deep joists and $L \times B = 5.00 \text{ m} \times 4.40 \text{ m}$ with 302 mm deep joists. The TJI 250 joists had Laminated Veneer Lumber (LVL) flanges of $b \times h = 45 \text{ mm} \times 38 \text{ mm}$ and 9.5 mm thick Oriented Strand Board (OSB) web (Figure 4.1) and were spaced at 400 mm with slight variations at the edges to accommodate the floor width (Figure 4.2). The joist ends were fixed to LVL rim boards (TimberStrand LSL) using one woodscrew of 3.9 mm × 55 mm per flange at both ends. The decking boards were connected to the joists using either woodscrews of 3.9 mm × 55 mm alone at 300 mm spacing or in combination with water resistant wood glue, complying with durability class D3 of BS EN 204:2001. The deck was either a 15 mm OSB layer whose sheets were staggered with their stiffer direction arranged perpendicular to the longitudinal floor direction or a 22 mm tongued and grooved particleboard P5 layer or a combination of both with the OSB layer at the bottom and the particleboard layer on top.

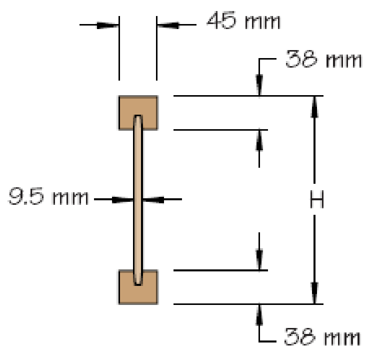


Figure 4.1: Properties of TJI 250 joist (TrusJoist 2003)

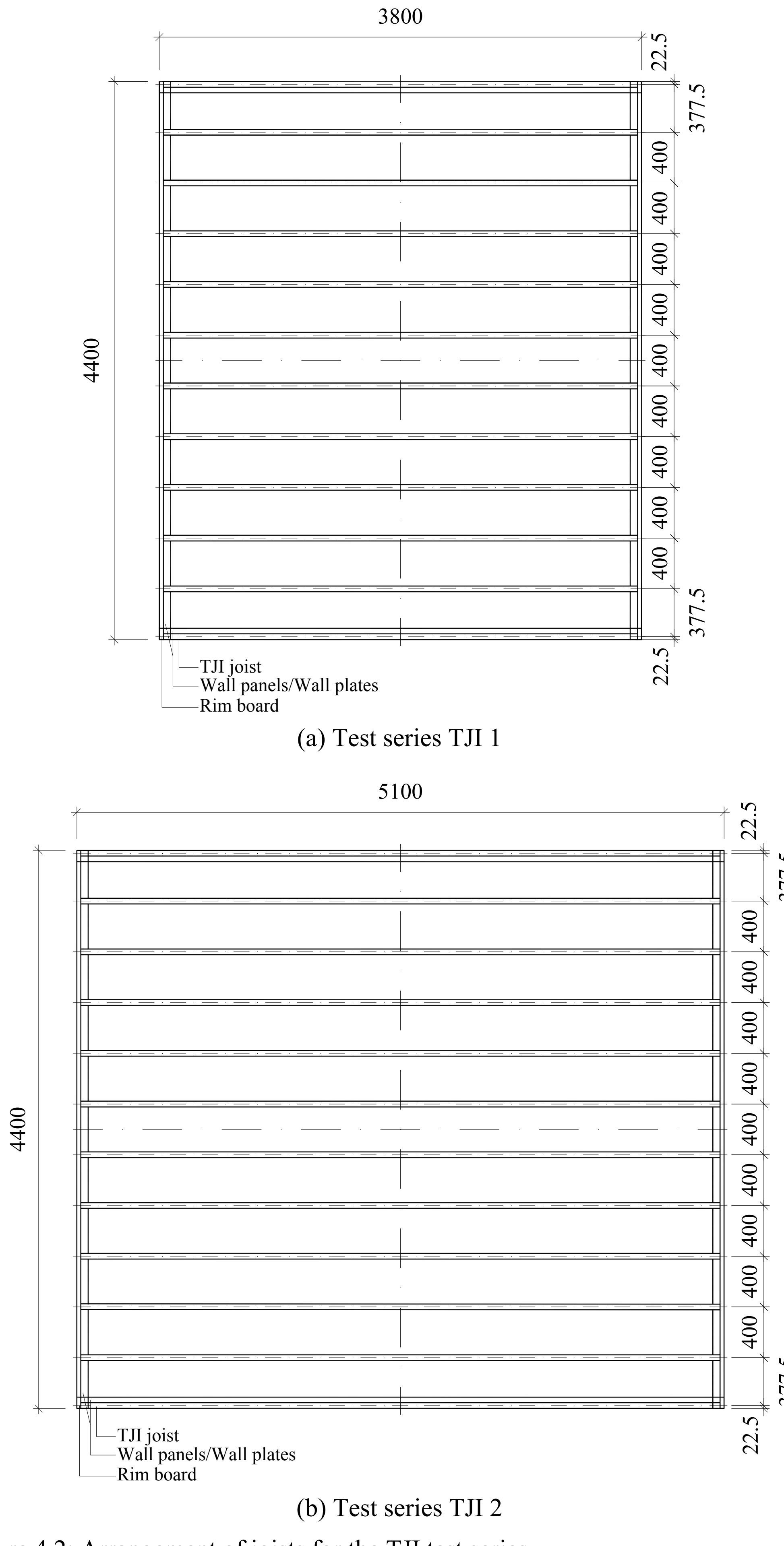
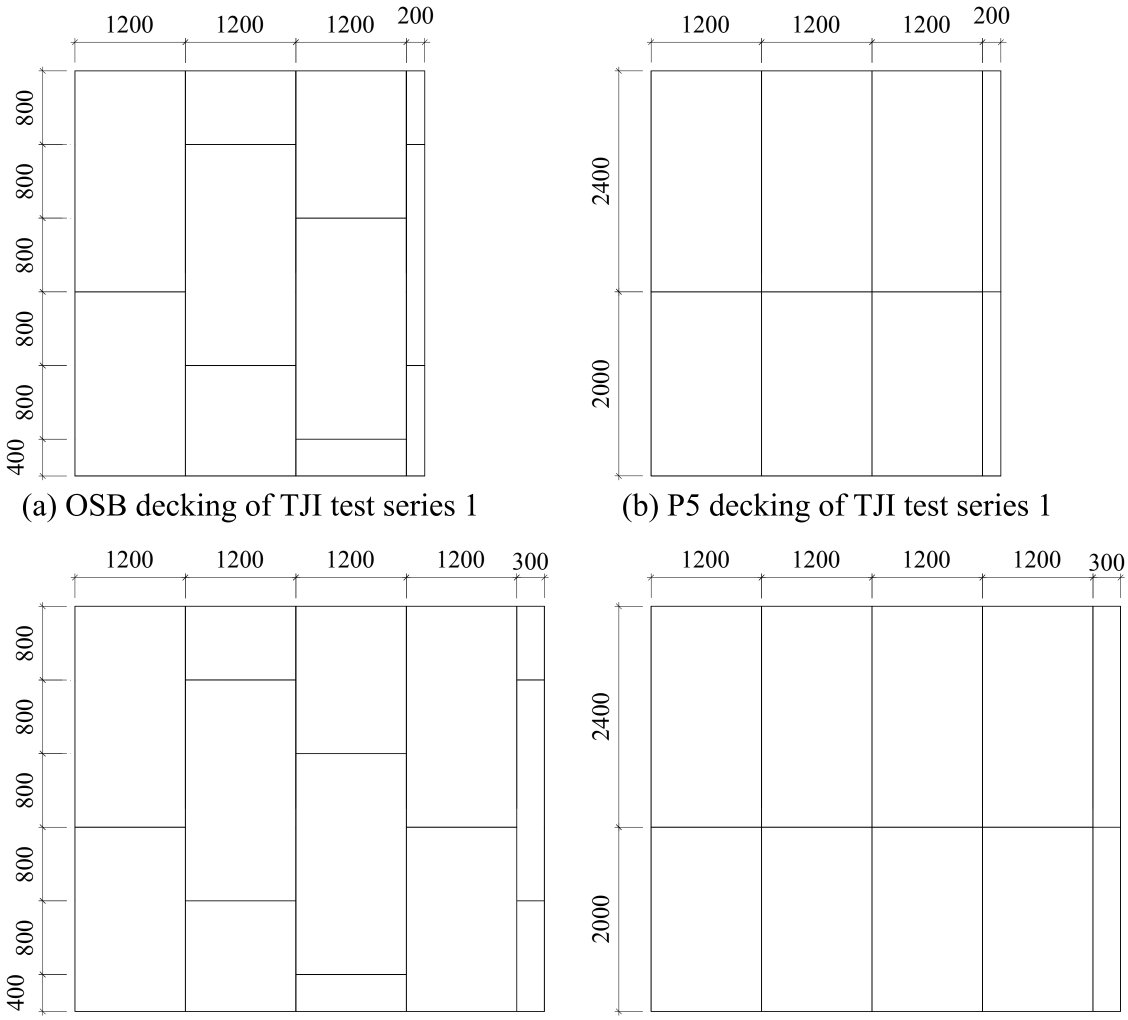


Figure 4.2: Arrangement of joists for the TJI test series

The latter was common Oregon Timber Frame LTD practice with the particleboard layer usually floating. For the test series, the second layer was not floating but screw-fixed to the underlayer and joists. The arrangement of the decking sheets is shown in Figure 4.3. The different decking types had different moduli of elasticity with $E_{0,m} = 4930$ N/mm^2 and $E_{90,m} = 1980$ N/mm^2 for the OSB and $E_m = 2700$ N/mm^2 for the particleboard types used, according to BS EN 12369-1:2001.



(a) OSB decking of TJI test series 1 (b) P5 decking of TJI test series 1

(c) OSB decking of TJI test series 2 (d) P5 decking of TJI test series 2

Figure 4.3: Layout of the floor decking sheets

To realise support conditions in the test environment as they can appear on sites for one-family timber-frame houses, timber wall plates of 38 mm × 89 mm were assembled to a rectangular frame and mounted to the concrete floor of the workshop. Wall panels of 300 mm height, constructed with solid timber post and beam members and OSB planks, were fixed to the two opposite wall plates of shorter length and laterally supported by timber struts.

The test floor, whose outer sides coincided with the outer sides of the frame, was simply supported on the top of the wall panels with no connection or screw-fixed to the supports with two equally spaced woodscrews of 3.9 mm × 55 mm between two adjacent joists along the rim boards. To support test floors along all sides, two further wall panels were squeezed in the existing gaps between the outer joist bottom flanges and the wall plates in longitudinal direction. They were then screw-fixed to the wall plates and to the transverse wall panels at the two edges. There was no connection between the floor and the added panels for simple support conditions and screw-fixing with a fastener spacing of 600 mm along the outer I-joist bottom flanges otherwise. The fastener spacing was adjusted at the edges to accommodate floor lengths. Details of support conditions for two-side supported floors are shown in Figure 4.4 and the construction of a flooring structure and corresponding supports in Figure 4.5.

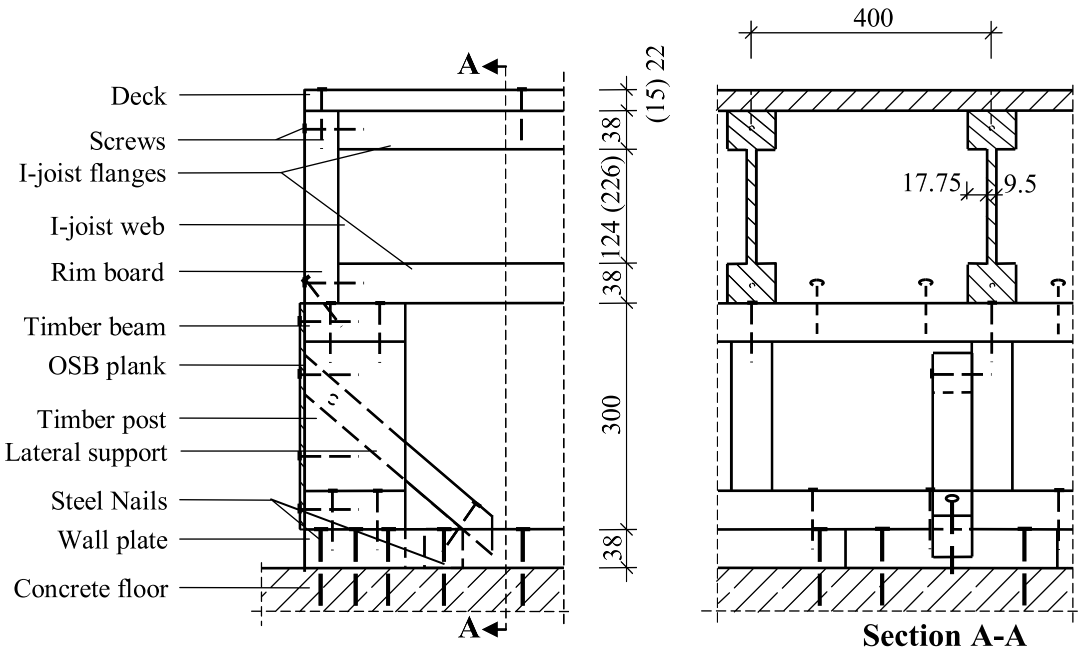


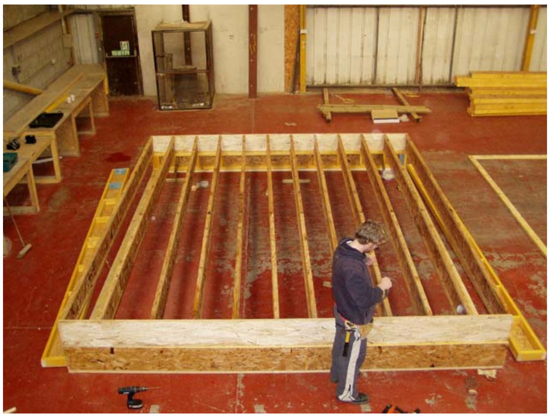
Figure 4.4: Details of support conditions of a floor supported along two sides, screw-fixed to the supports

The floors of Test series 1 were named as TJI floor 1, those of Test series 2 as TJI floor 2, plus the addition of a letter from A to O for identification of the structural detailing, which is presented in Table 4.1. Due to the use of adhesives to fix the deck to the joists in some cases, four complete samples, two for each test series, had to be constructed. This requirement was further incorporated in the testing programme so as to benefit the test progress. Floors with the structural detailing from A to G refer to one sample and from H to O to the other sample for each series. The structural details selected for investigations were either common Oregon Timber Frame Ltd practice (double layer deck) to explore the contribution of the applied method to dynamic response, or simple modifications to enhance support rigidity or composite action of deck and joists as options to be easily applicable in practice if successful.

(a) Wall plates assembled to frame

(b) Wall panels fixed onto the solid frame

(c) I-joists connected to rim boards and resting on wall panels



(d) Flooring structure without deck

(e) Complete floor with OSB deck

(f) Four floor samples

Figure 4.5: Floor construction and detailing

Table 4.1: Details of structures and modifications in TJI floor test series

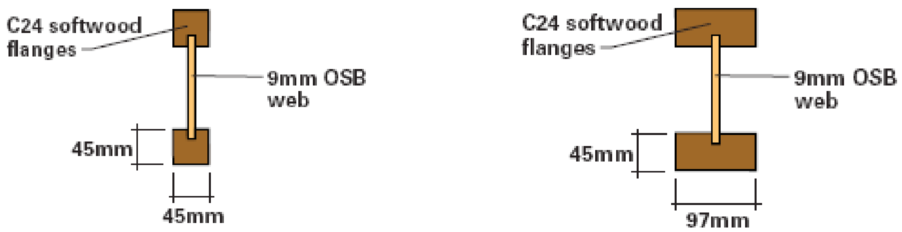
<table>
<tr><td colspan="6">Floor 1: Size: 3.70 m × 4.40 m; Joists: 45 (9.5) mm × 200 mm TJI 250;
Screw spacing : 300 mm
Floor 2: Size: 5.00 m × 4.40 m; Joists: 45 (9.5) mm × 302 mm TJI 250;
Screw spacing : 300 mm</td></tr>
<tr><th rowspan="2">Detailing / TJI floor</th><th rowspan="2">Deck</th><th rowspan="2">Fixing method
(Deck to joists)</th><th colspan="2">Supports</th></tr>
<tr><th>Condition</th><th>Sides</th></tr>
<tr><td>A</td><td>15 mm OSB</td><td>Screws</td><td>Simply</td><td>2</td></tr>
<tr><td>B</td><td>15 mm OSB</td><td>Screws</td><td>Screw-fixing</td><td>2</td></tr>
<tr><td>C</td><td>15 mm OSB</td><td>Screws</td><td>Screw-fixing</td><td>4</td></tr>
<tr><td>D</td><td>15 mm OSB</td><td>Screws</td><td>Simply</td><td>4</td></tr>
<tr><td>E</td><td>15 mm OSB
+ 22 mm P5</td><td>Screws</td><td>Screw-fixing</td><td>4</td></tr>
<tr><td>F</td><td>15 mm OSB</td><td>Glue + screws</td><td>Screw-fixing</td><td>4</td></tr>
<tr><td>G</td><td>15 mm OSB
+ 22 mm P5</td><td>OSB: Glue + screws
P5: Screws</td><td>Screw-fixing</td><td>4</td></tr>
<tr><td>H</td><td>22 mm P5</td><td>Screws</td><td>Simply</td><td>2</td></tr>
<tr><td>I</td><td>22 mm P5</td><td>Screws</td><td>Screw-fixing</td><td>2</td></tr>
<tr><td>J</td><td>22 mm P5</td><td>Screws</td><td>Screw-fixing</td><td>4</td></tr>
<tr><td>K</td><td>22 mm P5</td><td>Screws</td><td>Simply</td><td>4</td></tr>
<tr><td>L</td><td>22 mm P5</td><td>Glue + screws</td><td>Simply</td><td>2</td></tr>
<tr><td>M</td><td>22 mm P5</td><td>Glue + screws</td><td>Screw-fixing</td><td>2</td></tr>
<tr><td>N</td><td>22 mm P5</td><td>Glue + screws</td><td>Simply</td><td>4</td></tr>
<tr><td>O</td><td>22 mm P5</td><td>Glue + screws</td><td>Screw-fixing</td><td>4</td></tr>
</table>

4.2 Structural detailing of JJI joist floors and support conditions

4.2.1 Floor construction and structural and non-structural modifications

James Jones & Sons Ltd provided the material for the two JJI floor test series, which were conducted in the laboratory of Edinburgh Napier University. A total of twenty-nine floor configurations of $L \times B = 3.50 \text{ m} \times 2.44 \text{ m}$ were investigated. Test series 1 included the floors with 245 mm deep joists and Test series 2 those with 220 mm depth joists.

The JJI A joists had $b \times h = 45 \text{ mm} \times 45 \text{ mm}$ timber flanges of grade C24 and 9 mm thick OSB web (Figure 4.6(a)) and were initially installed at the common joist spacing for timber flooring structures of 600 mm. Also JJI D joists with $b \times h = 97 \text{ mm} \times 45 \text{ mm}$ timber flanges (Figure 4.6(b)) were used. The joist ends were fixed to glued laminated timber rim boards (grade: GL 24) using one woodscrew of 4.9 mm × 75 mm per flange at both ends. The decking boards were connected to the joists using woodscrews of 4.25 mm × 38 mm at 300 mm spacing. The deck was a 19 mm particleboard P5 layer with two boards of 2440 mm × 1200 mm and another board of adjusted width in-between.



(a) JJI A Joist (b) JJI D Joist

Figure 4.6: JJI joists used (James Jones & Sons LTD 2002)

Timber wall plates of 38 mm × 89 mm were assembled to a rectangular frame and mounted to the concrete floor of the laboratory. Two further wall plates of the same type were fixed onto the frame, one at each of the two opposite sides of shorter length. The test floor, whose outer sides coincided with the outer sides of the frame, was simply supported for three tests of Test series 2 by lying on the top of the wall plates with no connection. In the other cases the joist ends were screw-fixed to the supports at the bottom flanges with one screw on either side of the web. Hence, compared to the TJI floor test series, a different method was used to connect the floor to the supports to enhance end fixity. The initial floor configuration is illustrated in Figure 4.7, and details of the support conditions with screw-fixing are shown in Figure 4.8.

Test series 1 comprised the study of the dynamic responses of 16 test floors. Alterations were made to joist spacing, floor mass and the stiffness at sensitive locations. The concept of stiffening the floors at sensitive locations is described in Section 4.2.2. The alteration of joist spacing is in particular a modification of the global longitudinal flooring stiffness. The mass is another decisive property with regard to dynamic floor responses. The detailed testing programme of JJI floor Test series 1 is shown in Table 4.2.

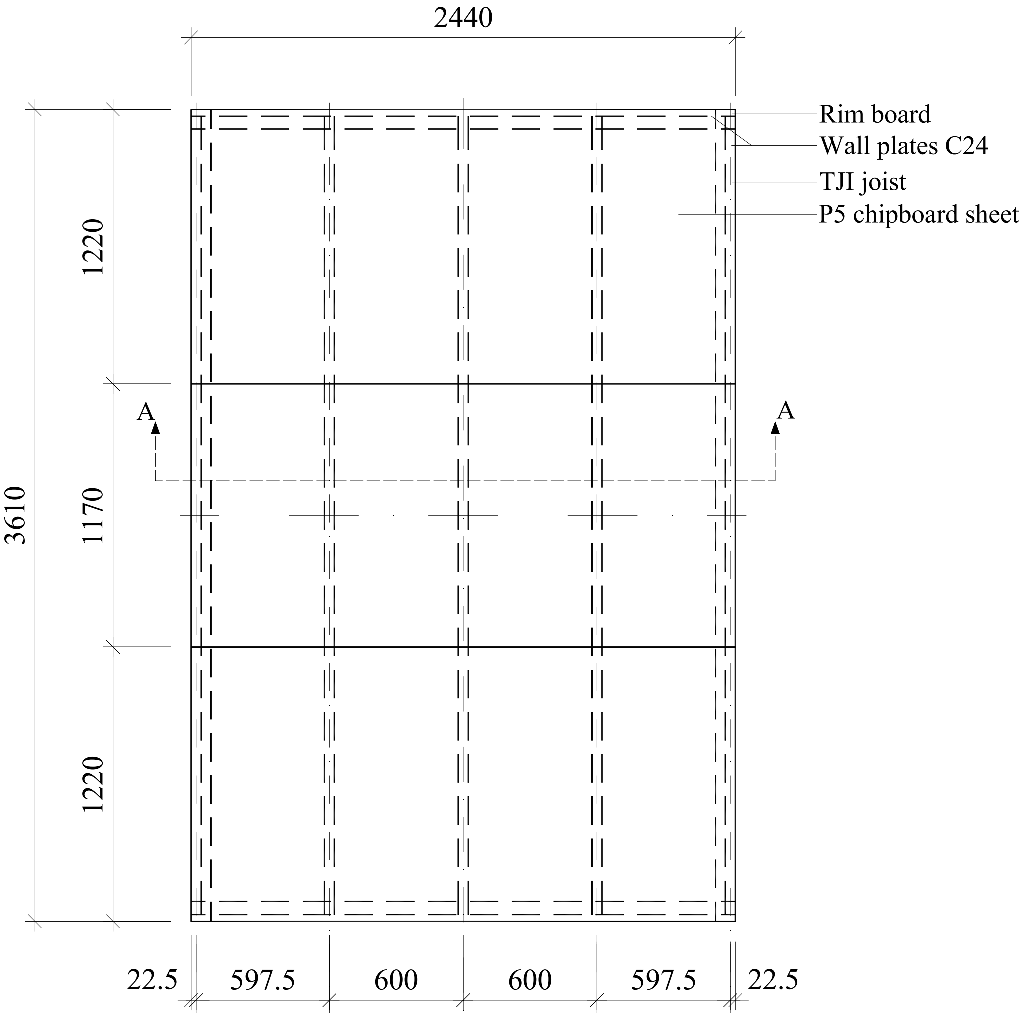


Figure 4.7: Details of initial configuration of JJI floor

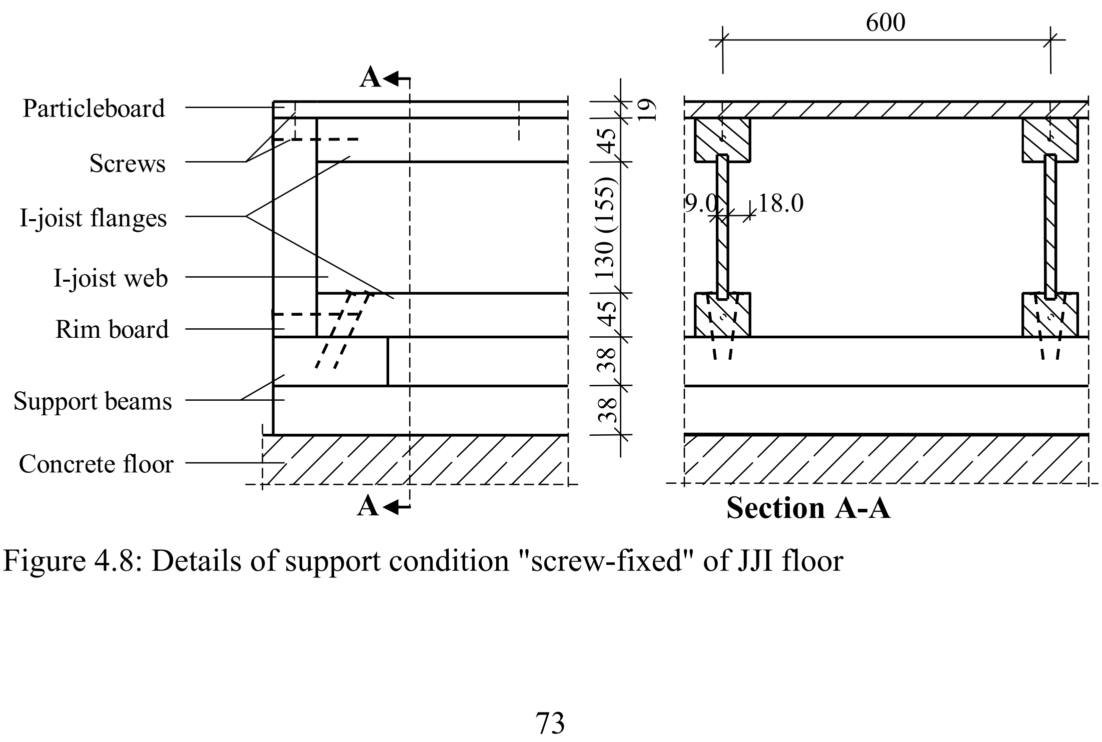


Figure 4.8: Details of support condition "screw-fixed" of JJI floor

Table 4.2: Details of structures and modifications in JJI floor test series 1

Floor 1: Size: 3.50 m × 2.44 m; Deck: 19 mm particleboard P5; Rim board: GluLam; Joists: 45 (9) mm × 245 mm JJI A joists (97 (9) mm × 245 mm JJI D joists); Fixing method: Screws; Screw spacing: 300 mm; Supports: 2 sides (screw-fixing, see Figure 4.8).

Detailing / JJI Floor	Joist spacing (mm)	Imposed load (kg)	Stiffer joists	
			Location	Joist type
A	600	-	-	-
B	600	30.1	-	-
C	600	55.1	-	-
D	600	-	Centre	JJI D
E	600	30.1	Centre	JJI D
F	600	55.1	Centre	JJI D
G	600	-	Centre	2 × JJI D
H	600	30.1	Centre	2 × JJI D
I	600	55.1	Centre	2 × JJI D
J	600	-	Centre	2 × JJI A
K	600	30.1	Centre	2 × JJI A
L	600	55.1	Centre	2 × JJI A
M	600	-	Edges	2 × JJI A
N	600	-	Centre + edges	2 × JJI A
O	400	-	-	-
P	300	-	-	-

Test series 2, which is detailed in Table 4.3, contained the investigation of dynamic response of 12 flooring systems with varying properties plus retesting Floor A as the final configuration to investigate possible impacts of reusing floor material and screw holes on the vibration performance. As for Test series 1, stiffness at sensitive locations, floor mass and joist spacing varied for the investigations. Also the effects of I-joist blocking (Figure 4.9) as increased stiffness in transverse direction and of varied support conditions such as simple supports or screw-fixing the floor to the supports were studied. Some results in both test series can be compared for examining the efficiency of varied joist depth for such modification in stiffness.

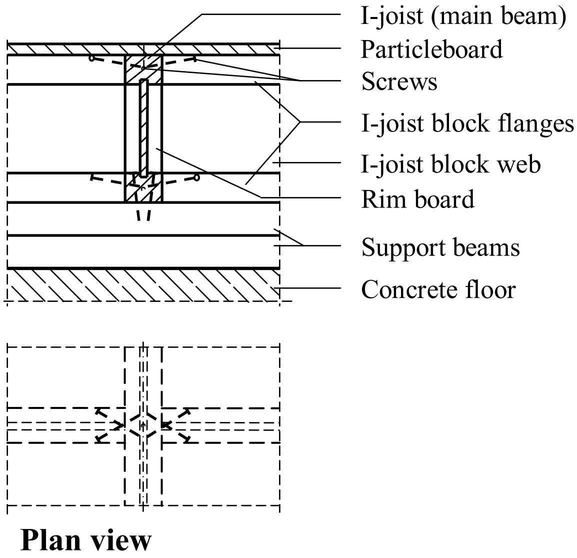


Figure 4.9: Connection of I-joist blocking

The floors of Test series 1 were named from A to P. The floors of Test series 2 got the same letter if the structural detailing, apart from the different joist height, was the same. The other floors of Test series 2 were named from Q-U. The retested floor had the same label as the original floor plus two stars. To clearly distinguish between the two test series, the floors of Test series 1 were named as JJI Floor 1, those of Test series 2 as JJI Floor 2 in front of the aforementioned letters.

4.2.2 Concept of using double joists for local stiffening

From subjective site observations and consequential numerical studies (on four-side supported floors), Smith and Chui (1988a) found that a very flexible joist in floor centre could affect the vibration performance of light-weight floors in a highly negative way, in particular for the first vibration mode. Ohlsson (1994) reported many complaints about floor vibration associated with large movements for floor areas close to unrestrained edges. Studying the typical mode shapes in transverse direction for two-side supported timber flooring structures can thus help to identify highly sensitive locations with large modal displacements (see Figure 4.10). Therefore, it could be beneficial to stiffen especially the identified soft spots so as to lower the modal amplitudes and the static deflections at these locations and to raise the natural frequencies in general. However, as indicated in the conducted literature review about the possible enhancements in design (Section 3.2), an investigation of targeted structural modification at dynamic soft spots as described in this section were not observed to be carried out yet.

Table 4.3: Details of structures and modifications in JJI floor test series 2

Floor 2: Size: 3.50 m × 2.44 m; Deck: 19 mm particleboard P5; Rim board: GluLam; Joists: 45 (9) mm × 220 mm JJI A joists (97 (9) mm × 220 mm JJI D joists); Fixing method: Screws; Screw spacing: 300 mm; Supports: 2 sides						
JJI Floor \ Detailing	**Joist spacing (mm)**	**Imposed load (kg)**	**Stiffer joists**		**Transverse blocking**	**Support**
			Location	**Joist type**		
A	600	-	-	-	-	Screw-fixing
G	600	-	Centre:	2 × JJI D	-	Screw-fixing
J	600	-	Centre:	2 × JJI A	-	Screw-fixing
M	600	-	Edges:	2 × JJI A	-	Screw-fixing
N	600	-	Centre + edges:	2 × JJI A	-	Screw-fixing
O	400	-	-	-	-	Screw-fixing
P	300	-	-	-	-	Screw-fixing
Q	600	-	-	-	-	Simply
R	600	30.1	-	-	-	Simply
S	600	55.1	-	-	-	Simply
T	600	-	-	-	Centre	Screw-fixing
U	600	-	-	-	3rd points	Screw-fixing
A**	600	-	-	-	-	Screw-fixing

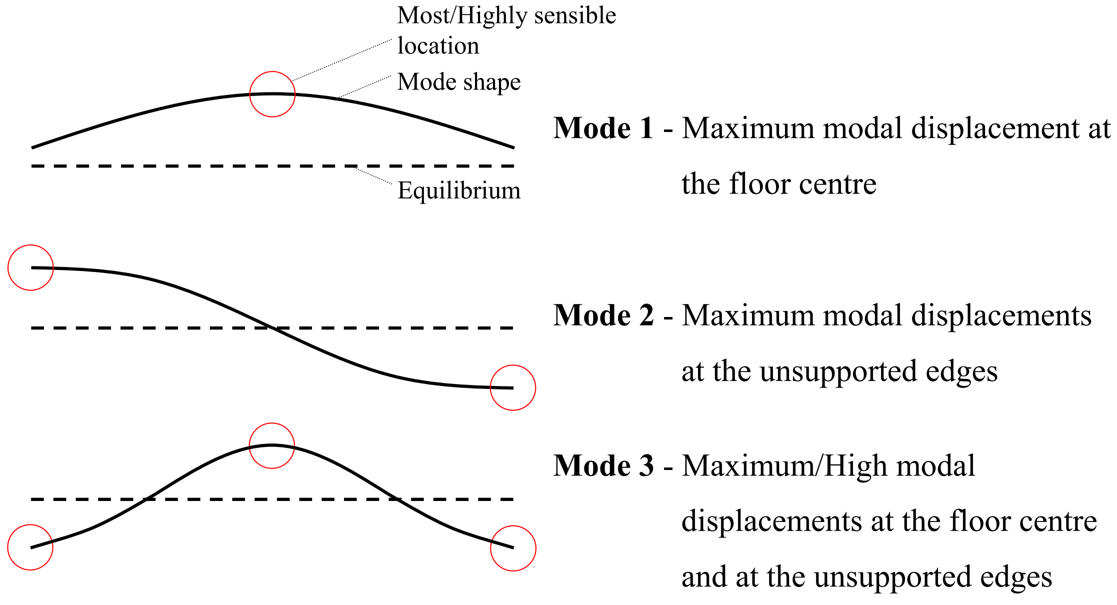


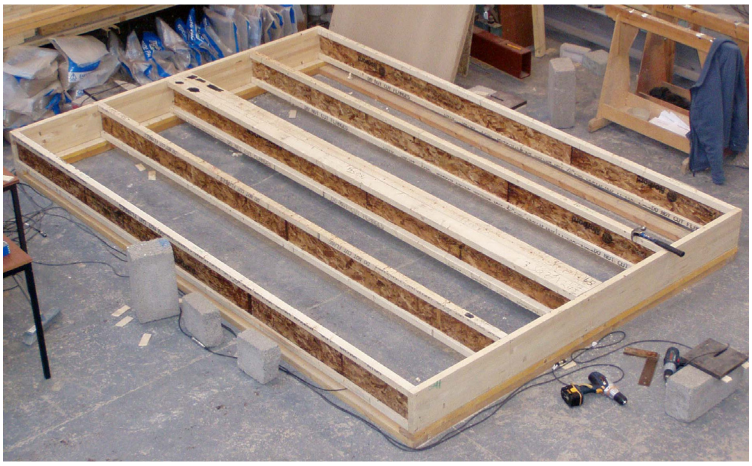
Figure 4.10: First three principal vibration mode shapes in transverse direction for one-way spanned two-side supported rectangular floors

The concept of increasing the stiffness at dynamically sensitive locations was presented at WCTE 2008 (Weckendorf et al. 2008a) and is very simple and hence easy to apply. Instead of using single I-joists evenly distributed over the whole structure, two I-joists together are placed at the floor areas where high modal displacements may occur. Positioning double joists along the central line was expected to benefit especially the first and third modes of vibration, and placing double joists at the edges could particularly enhance the second and also the third mode. Constructing the floor with double joists along the central line and edges could thus benefit at least the first three principal modes simultaneously by increasing natural frequencies, lowering the modal amplitudes and additionally reducing static point load deflections at the locations of the double joists and neighbouring joists. Furthermore, I-joists with increased flange width could be used to amplify the described effects.

These measures are also in line with suggestions by Ohlsson (1982). To improve the vibrational behaviour of light-weight floors due to transient impulsive loading, he proposed to raise the damping coefficient and modal mass. Damping is not easily controlled. However, Ohlsson mentioned four different ways to increase modal mass whereas he added that two of them could lead to very bad results, and therefore great care would be required for their use. "*To change the mass distribution for the floor so that more of the mass will be located near anti-nodes*" and "*to change the mode shape by local stiffeners etc*" were the other two methods. Whereas Ohlsson referred to transverse beams for the latter case, these two steps could be purposefully tackled by using double joists at the location of anti-nodes. This was experimentally investigated in the JJI test series. The reference floor was constructed with five JJI A joists at a spacing of 600 mm. For stiffening the sensitive locations, initially the centre JJI A joist had been exchanged by a single JJI D joist for the floors JJI D, E, F, and based on the reference floor, double joists were introduced as follows (see also Table 4.2 and Table 4.3):

- Double JJI A joists along the central line only (JJI floors J, K, L)
- Double JJI D joists along the central line only (JJI floors G, H, I) (Figure 4.11)
- Double JJI A joists along the edges only (JJI floors M)
- Double JJI A joists along the edges and the central line. (JJI floors N)

Detailed support conditions for the floor with double joists are shown in Figure 4.12.



(a) Flooring structure with two JJI D joists in the centre and four separated JJI A joists

(b) Detail of double joist at the supports

Figure 4.11: Construction of Floor JJI 1G

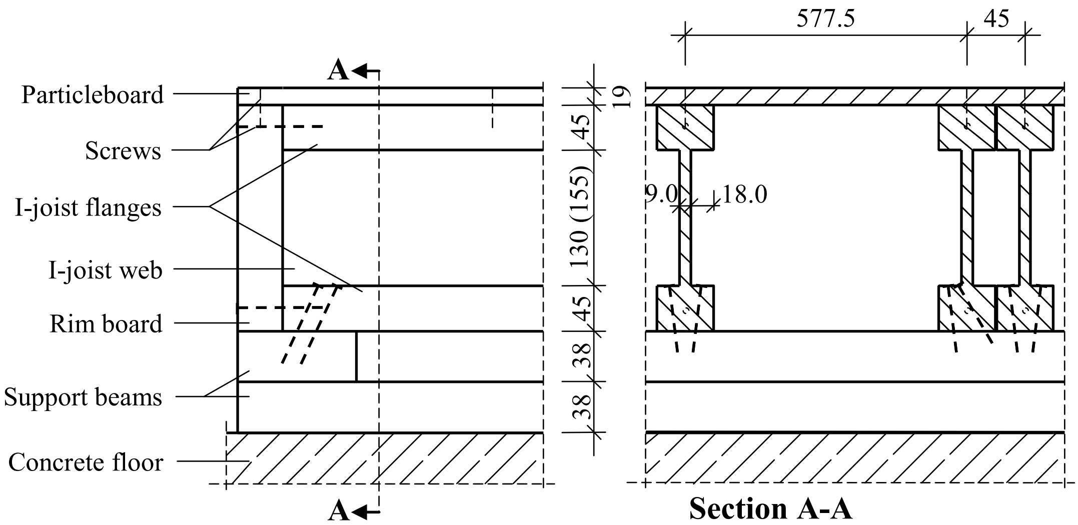


Figure 4.12: Detailed end support conditions of floor with double joists

4.3 Structural detailing of metal-web joist floors and support conditions

The metal-web joist floor test series, comprising nine floor configurations of the size $L \times B = 5.150\text{ m} \times 4.897\text{ m}$, was conducted in the laboratory of Edinburgh Napier University. All floors were constructed with metal-web joists (open-web joists) of 47 mm × 97 mm TR26 solid timber top and bottom chords and MS250 steel webs with an overall joist depth of 254 mm. The joists were connected to C16 solid timber rim boards of 47 mm × 222 mm with three woodscrews of 5.0 mm × 100 mm, flush with the top surface. The 22 mm tongued and grooved particleboard P5 decking sheets of 2400 mm × 600 mm were fixed to the metal-web joists and to the rim boards with 4.1 mm × 50 mm woodscrews at a spacing of 300 mm.

The support structure for the floors comprised two 2-ply chord girder walls of $h \times B = 1.2\text{ m} \times 5.0\text{ m}$ and five 45° triangular outriggers at each end to support the two walls. The top chords of the girder walls were manufactured from 47 mm × 147 mm solid timber and the bottom chords and bracings were from 47 mm × 72 mm solid timber. The outriggers had a cross section of 35 mm × 72 mm. All the chord girder walls and outriggers were directly connected to the concrete floor. The tested floors were fixed down to the support structures with two woodscrews of 4.9 mm × 75 mm per joist end. The metal-web joists connected to the support structure are illustrated in Figure 4.13.


(a) Metal-web joists on supporting truss

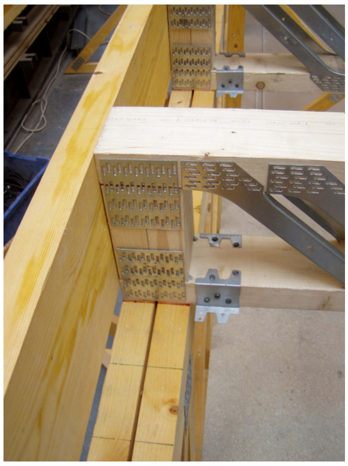

(b) Metal-web joists connected to support and rim board

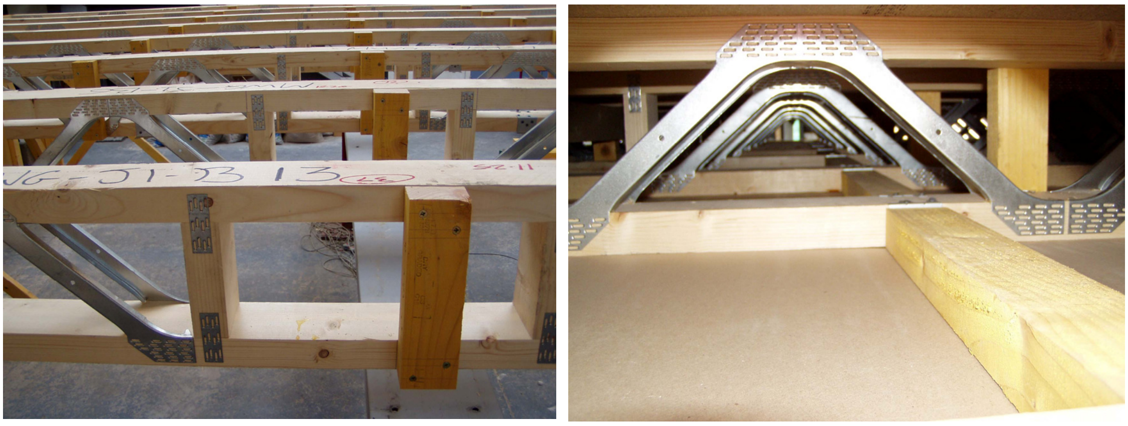
Figure 4.13: Metal-web joists connected to the support structure

The effects of using strongback and ceiling and of reducing the joist spacing were examined as detailed in Table 4.4. Strongback implies those timber beams, which were employed as transverse stiffeners of the test floors. Dimensions, location and material of the strongback varied with the tests. Strongback noggings of C16 timber with a cross section of 47 mm × 72 mm were cut 1 mm short of joist depth and fixed to both chords of each joist with two woodscrews of 5.0 mm × 100 mm (Figure 4.14(a)). The strongback elements were inserted through the gaps of top and bottom chords of the joists, then fitted tightly to the lower side of the top chord and fixed to the noggings with three woodscrews of 5.0 mm × 100 mm.

Table 4.4: Details of structures and modifications in metal-web joist floor test series

MWJ Floor	Size: 5.150 m × 4.897 m; Joists: 47 mm × 254 mm Metal web joists; Deck: 22 mm particleboard P5; Rim board: C16 timber; Fixing method: Screws; Screw spacing: 300 mm; Supports: 2 sides		
	Joist spacing (mm)	**Strongback**	**Ceiling**
A	600	-	-
B	600	47 mm × 147 mm TR26 at mid-span	-
C	600	45 mm × 147 mm Kerto at mid-span	-
D	600	35 mm × 97 mm TR26 at 1/3 spans	-
E	600	35 mm × 97 mm TR26 at mid-span	-
F	600	35 mm × 97 mm TR26 at mid-span	12.5 mm wallboard
G	400	-	-
H	400	35 mm × 97 mm TR26 at mid-span	-
I	400	35 mm × 97 mm TR26 at mid-span	12.5 mm wallboard

Ceiling noggings of C16 timber with a cross section of 47 mm × 72 mm were fixed to the joists at the location of plasterboard joints (Figure 4.14(b)) using Cullen UZ/47 clips and 3.75 mm × 30 mm nails. The 12.5 mm thick ceiling of 2400 mm × 1200 mm wallboard was fixed to the noggings and joists with 3.5 mm × 42 mm drywall screws at a spacing of 150 mm along the perimeter of the sheets and at a spacing of 230 mm on internal joists. Ceiling and the decking layouts are shown in Figure 4.15. The decking layout was the same for all floors, and the ceiling layout was the same for the two floors it was used for. These test floors are named as MWJ floor (metal-web joist floor) plus a letter from A-I for identification of the structural detailing.

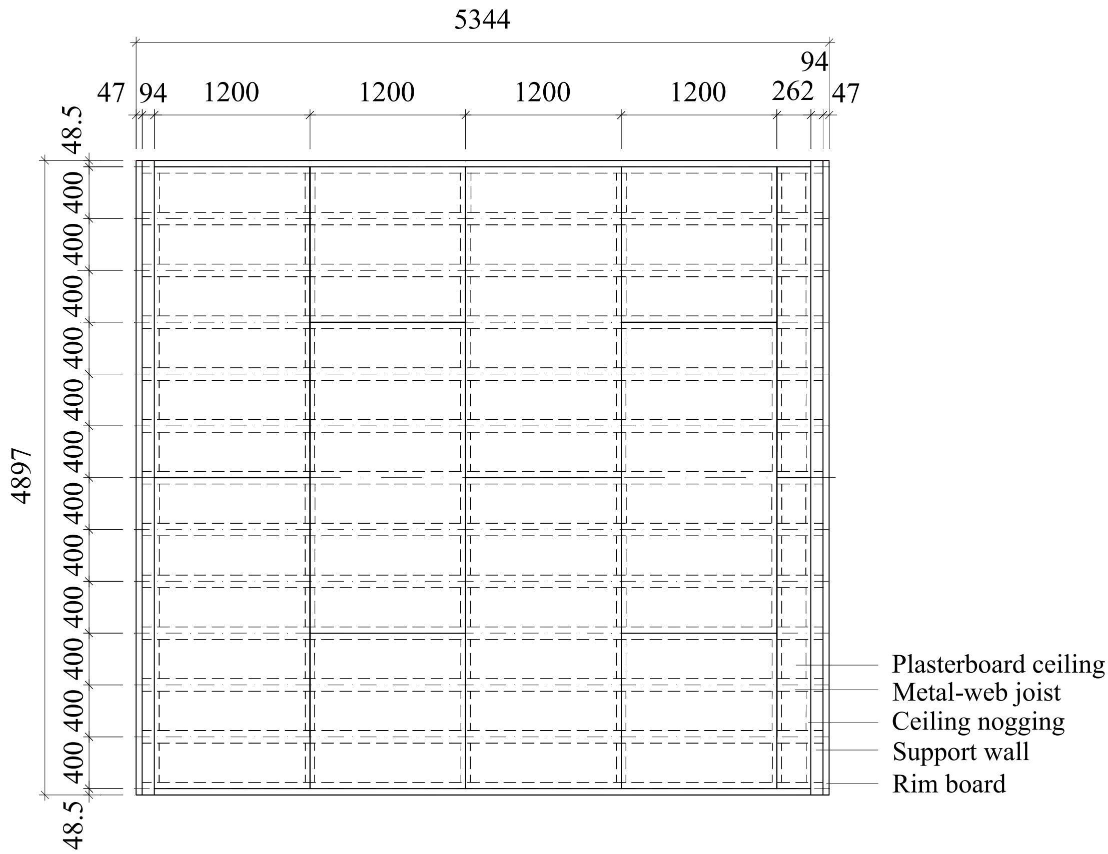


(a) Strongback noggings at joist side (b) Ceiling noggings between adjacent joists

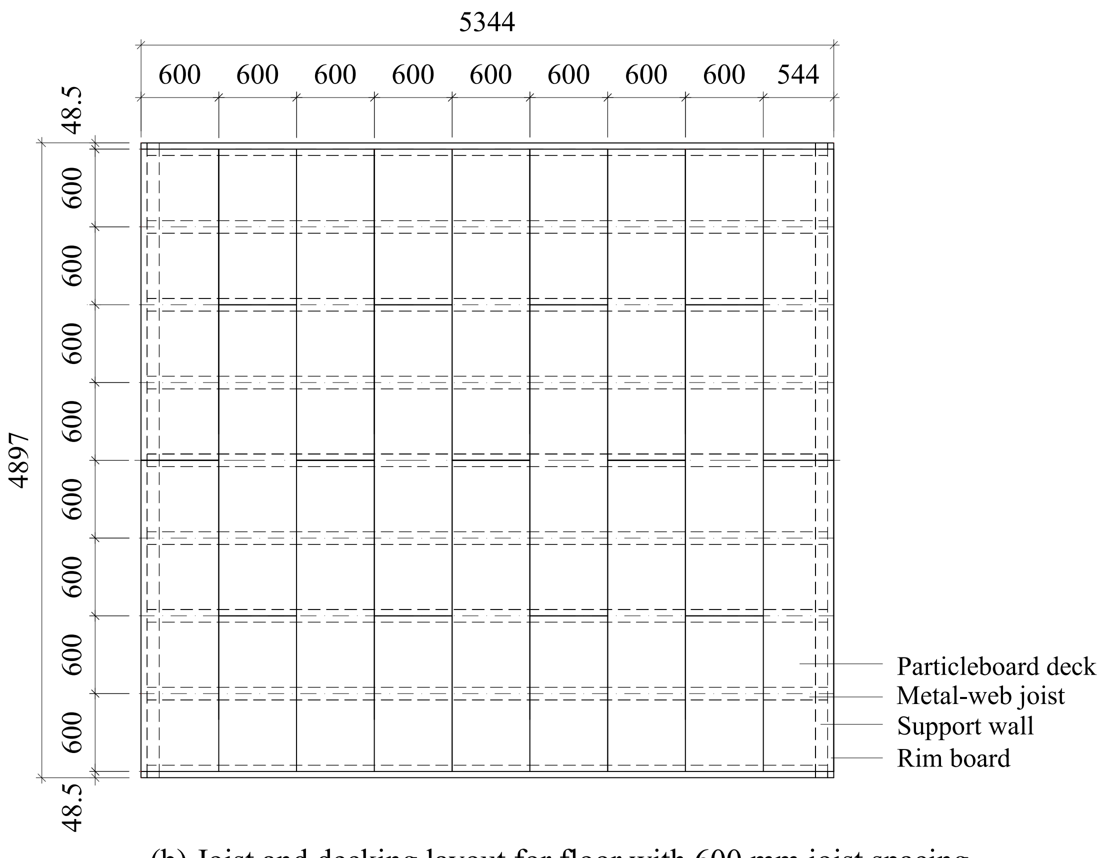
Figure 4.14: Noggings installed to joists for different floor configurations

4.4 Deflection tests

The highest deflection of rectangular floors can usually be expected at mid-span. In transverse direction, two aspects need to be considered to identify the weakest point. Due to the decking sheets, the load is partially transferred to adjacent joists. The magnitude of the load-sharing effect depends on the stiffness of deck and joist elements and the degree of composite action. The other aspect is whether the floor is supported along two or more sides. As the load is approaching the area close to one of the edges, the deflection at the point of application can be expected to be decreasing for floors supported along all edges since the outer joists are directly supported and to be increasing for floors supported at only the joist ends as the load sharing effect is lowered due to the reduced effective width at one side. Although the point load deflection at the edges can be higher, in literature it is often referred to the deflection at the floor centre. According to EC5-1-1, however, the maximum deflection at any point on the floor needs to be below the design limit. As the outer joists of relatively wide floors may not contribute to a load-sharing effect if the floor is loaded in the centre, the deflection measured at this point of application may be similar and therefore representative for both cases: floors supported along two sides and along four sides. In the conducted experimental work, the deflection was usually measured at the floor centre but in some cases additionally at mid-span of other floor joists. The details of the deflection tests, which were carried out differently for the three main test series, are explained in the following Sections 4.4.1 - 4.4.3.



(a) Joist and ceiling layout for floor with 400 mm joist spacing



(b) Joist and decking layout for floor with 600 mm joist spacing

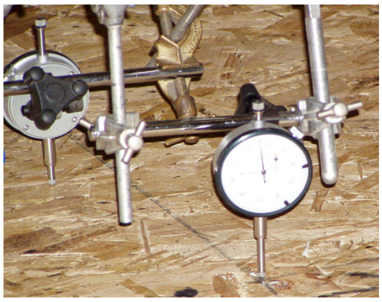
Figure 4.15: Layout of joist, deck and ceiling arrangements of metal-web joist floors

4.4.1 Deflection tests of TJI floor test series

The flooring structures of the TJI floor test series were all constructed with even numbers of joists as can be seen from Figure 4.2. This means that there was no joist running along the central line. It was decided to divide the load onto the mid-span of the two joists next to the centre. To induce deflection, a load of 1 kN, from ten steel sections of 0.1 kN each, was applied. This test method was maintained for the TJI floor test series.

A dial gauge with an accuracy of 0.01 mm fixed to a scaffold post running across the floor was placed at the centre point of the floor surface. Dial gauges were also installed at the middle point of each edge to examine whether there were noticeable movements at the edges when the floor was loaded (Figure 4.16(a)). A secondary dial gauge installed close to the centre (Figure 4.16(b)), and test repetitions when the floor had recovered after unloading, served as inspection measures to ensure stable results. The test repetitions also allowed using averaged results for the analysis.

(a) Dial gauges in the centre and at the edges



(b) Inspection measure behind main dial gauge

Figure 4.16: Position of dial gauges on the floor

Readings were taken from the dial gauges before and after loading the structure. The difference from the two readings of each dial gauge was calculated. The net deflection of the floor was then determined by adjusting the measurement from the floor centre by considering the movement at the joist ends. The measured deflections were compared to calculations following the design equations of the standard (UK NA to EC5-1-1) and published in Weckendorf et al. (2006), which showed passable correlation of predictions and measured results for the larger floors but poorer correlation for the smaller ones. This could have been caused by the loading case of distributing the load directly onto the location of two joists, which was slightly differing from the loading case mentioned in the UK NA to EC5-1-1.

4.4.2 Deflection tests of JJI floor test series

All test floors were loaded at the centre of the floor. For some tests the load was thereafter shifted to mid-span of the joists close to one of the edges: in Test series 1 for all floors without the added dead weight and in Test series 2 for the floors with double joists or I-joist blocking and for Floor JJI 2 A. For the floors with shortened joist spacing of Test series 1 also the second joist next to the edge was loaded separately at mid-span (Figure 4.17). In case of Floor JJI 1 A, the load was finally shifted to mid-span of one of the unsupported edges. The reason to apply the load on other than the central points was to study how the deflection varies when the load approaches one of the unsupported edges and to what degree neighbouring joists would be affected due to the structural modifications with double joists.

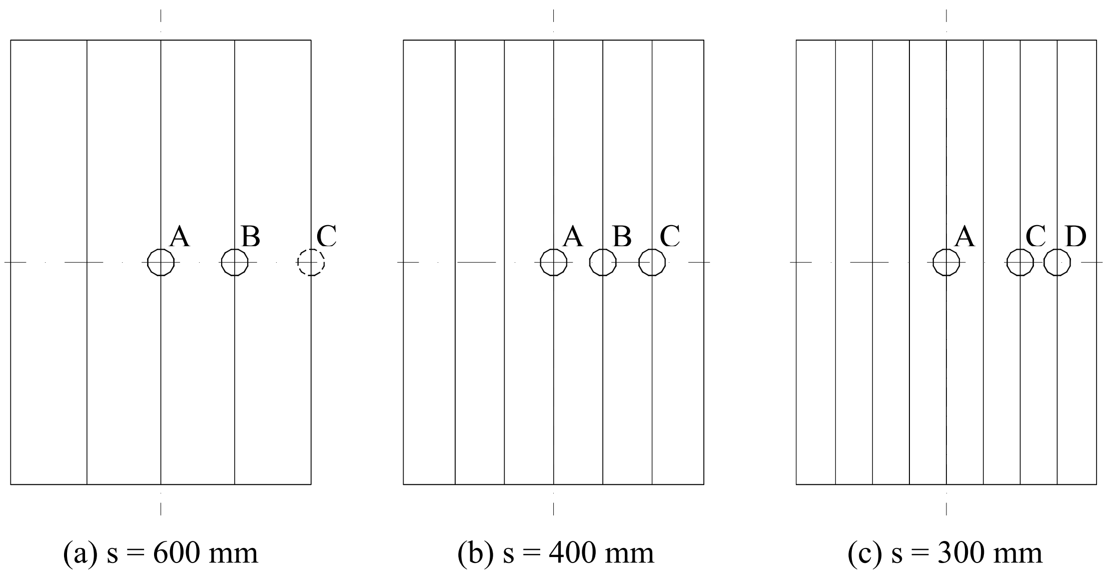


Figure 4.17: Load points A to D of floors JJI 1 with different joist spacing

A load of 1 kN, composed of steel plates, was applied to induce the deflection. Dial gauges were placed at the floor centre and at the middle point of each edge to measure the deflection. The positions of dial gauges at the floor centre and at the supported edges were adjusted when the deflections at mid-span of other than the centre joists were measured. To assure accurate results, the deflection tests were repeated at least three times when the floor recovered after unloading. Averaged values of three deflection measurements per location were used in the analysis.

4.4.3 Deflection tests of metal-web joist floor test series

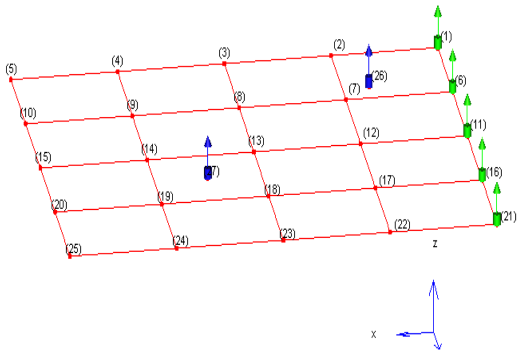
An extensive investigation of point load deflection was carried out on the flooring structures of this test series. The floors were loaded at mid-span of each joist. In accordance with the loading case in the UK NA to EC5-1-1, a load of 1 kN, composed of steel plates, was used to induce the deflection. Linear voltage displacement transducers (LVDTs) with an accuracy of 0.005 mm were installed at mid-span of each joist (Figure 4.18). For Floors A to F LVDTs were also used at each corner and at the middle of the two supported edges. For the Floors G to I one LVDT per supported edge was used whereas the transducer moved from position to position in accordance with the load. This was necessary as the number of LVDTs was limited. The deflection was measured three times at each LVDT location. For the joist ends where no transducers were used, the deflection values were obtained by interpolation. Averaged deflection values were used in the analysis.

Figure 4.18: LVDTs at mid-span

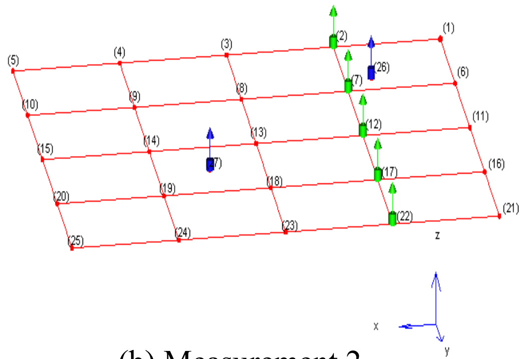
4.5 Vibration testing and modal parameter estimation

The dynamic testing consisted of an output-only modal analysis, which was carried out on all flooring structures to obtain the modal parameters, including modal frequencies, damping and shapes. The equipment used for the dynamic tests consisted of a TEAC LX-10/10L data recorder (see TEAC Corporation (2006)), Pinocchio vibraphones of the type A 150 X Vertical (see Pinocchio Data Systems (2004) - sensitivity may vary slightly for individual sensors) for measuring velocity, a laptop with the ARTeMIS Testor and Extractor modal analysis software package and a brush or a trolley.

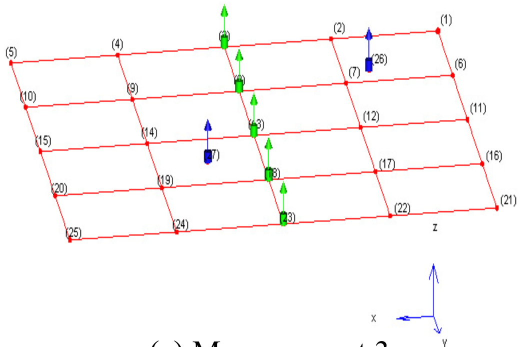
A grid with equally distributed node points was drawn on the floor surfaces. The node points served as the measurement points the sensors were attached to. The sensors were furthermore connected to the data recorder, which was connected to the laptop. The grid was also drawn to scale in the ARTeMIS Testor software programme, which was used for the test conduction. Figure 4.19 shows the measurement configuration for the MWJ floor test series where the sides (1)-(21) and (5)-(25) reflect the supported edges. Five roving sensors were used to cover all 25 measurement points within 5 measurements (see green arrows in Figure 4.19). Additional two sensors were placed as references at two different steady locations where all vibration modes of interest could be detected (see blue arrows in Figure 4.19). The configuration was fundamentally the same for the JJI and MWJ test series. The grid was divided into 5 × 5 and 6 × 5 points for the two-side supported floors and 5 × 7 and 6 × 7 for the four-side supported floors in the TJI Test series 1 and 2 respectively. The selected number of measurement points allowed for identifying all modes of interest after signal processing. Furthermore, the number and location of points on the surface provided a good representation of the floor response to reliably quantify the modal frequencies and damping. The time for each measurement within a full test was constant, taken as 100 seconds (minimum).



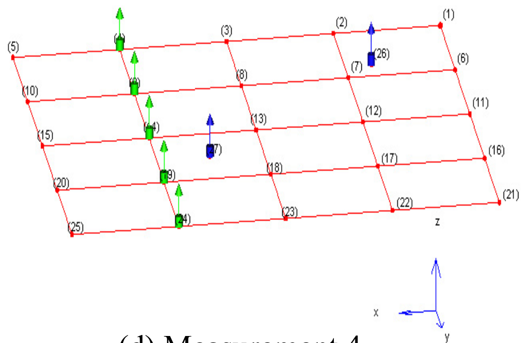
(a) Measurement 1



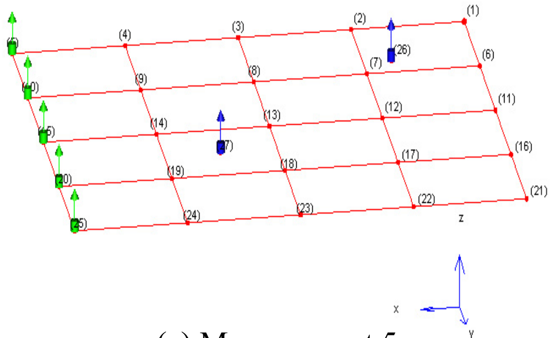
(b) Measurement 2



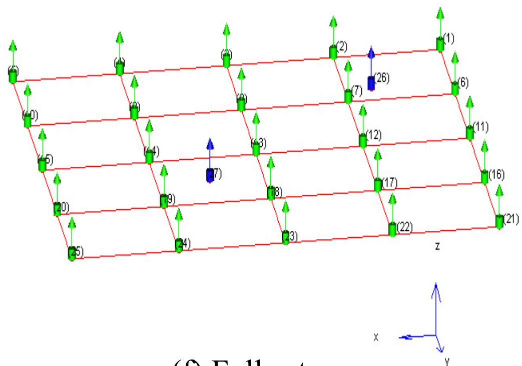
(c) Measurement 3



(d) Measurement 4



(e) Measurement 5



(f) Full set

Figure 4.19: Transducer configurations for measurements of MWJ floor test series

Only four sensors were available at the beginning of the test accomplishment regarding the TJI floor test series. This number increased gradually due to further purchased sensors as the tests progressed. The number of reference sensors was kept constant for all tests but the number of roving sensors adjusted under consideration of the number of available sensors.

A continuous excitation method was adopted, with the input magnitude unknown. Suitable excitation methods had been identified by trials prior to the test conduction. For the TJI and JJI floor test series each floor was excited by brushing the whole floor surface while walking around the floor or over a bridge above the structure. For the MWJ floor test series a trolley with a squared wooden board of 350 mm × 350 mm, four small wheels and a wooden handle was used to excite the floor. The trolley was loaded with bagged aggregates, forming a total weight of 5 kg. This trolley was pushed up to the central line of the floor and pulled back to the floor edge while moving it from one side of the floor to the other side, covering the whole floor area, so as to excite the different modes of vibration. The vibration in vertical direction was of interest. The Pinocchio Vibraphones transformed the vibrational motion of the floors into electrical signals, which were recorded by the data recorder.

Based on the assumption that the structural system under test would be excited by white noise, all modes were excited equally and the output spectrum contained full information of the structure. In practice, however, white noise excitation cannot be expected. The real excitation possesses rather a spectral distribution. In the combined (response) spectrum the physical modes of the structure and the noise modes due to excitation are present. Further non-physical modes are usually added due to computational and measurement noise and possibly harmonics due to rotating parts. The modal model of the structural system needs to be extracted from the estimated model of the combined system (Andersen 2005).

The ARTeMIS Extractor software includes three modal analysis methods: the Frequency Domain Decomposition (FDD), the Enhanced FDD (EFDD) and the Stochastic Subspace Identification (SSI). The FDD is a frequency domain analysis technique, the EFDD a combination of frequency and time domain analysis, and the (data-driven) SSI a pure time domain identification technique (Brincker et al. 2001b; Brincker and Andersen 2006). The FDD is the simplest one, without the estimation of damping.

A total of thirty-three flooring systems were analysed with both EFDD and SSI, including twelve TJI floors, twelve JJI floors and all nine metal-web joist floors. The application of both methods allowed for verification of the results. The variation in natural frequencies of the same mode was rather insignificant comparing the analysing methods. The damping ratios identified from the EFDD regarding the first vibration mode were found to be normally higher than those estimated from the SSI for the floors constructed with I-joists. For higher modes, the damping results from the two methods were usually closer. Therefore, under consideration of the capabilities of the analysing techniques, the SSI method had been selected for analysis of all test data towards representation of the results in this thesis. The signal processing is described below with focus on the SSI. However, essential information regarding the signal processing with the EFDD is provided for the sake of completeness.

In the EFDD, the measured signals are processed by a Discrete Fourier Transform (DFT) to obtain the spectral densities in the frequency domain and an inverse DFT is applied to SDOF spectral density functions, determined from the spectral peaks, for modal parameter estimation in the time domain. The frequency is determined by the number of zero crossings of the free decay time domain function, the normalised SDOF auto-correlation function. The damping is determined from the logarithmic decrement of the auto-correlation function (Brincker et al. 2001a).

Leakage is a bias error inherent with a DFT due to the finite length of the data record. Leakage is harmful to the accuracy of the results, especially to damping estimates. This leakage error can be reduced by increasing the frequency resolution. In turn, using lower frequency resolution results in more averages, which reduces noise influences (Harris and Piersol 2001; Zhang et al. 2002b). As lower frequency resolution tends to result in over-estimation of the damping, a compromise needs to be found by performing a number of signal analyses of the same data so that modal parameters can be estimated with optimised degree of accuracy. The Hanning window, a weighting function the signals are multiplied with, is automatically applied to the data in the ARTeMIS Extractor to reduce leakage due to the DFT. More detailed information on the FDD techniques can be found in the publications by Brincker et al. (2001a, 2001b)

In the SSI there are the two classes: data-driven and covariance driven. The SSI method used in this study to determine the modal parameters is data-driven. The parametric models are fitted directly to the measured time responses. This direct application to the raw time data allows for less biased results, e.g. the leakage problem can be avoided. In subspace identification QR-factorisation (leading to significant data reduction), singular value decomposition (SVD) (to reject noise, which is assumed to be represented by the higher singular values), least squares and eigenvalue decomposition are the steps from linear algebra applied to measurements for estimating the modal parameters (Andersen et al. 1999; Peeters and De Roeck 1999). It is beyond the scope of this thesis to explain the full details of the SSI but some basic information is summarised below. For further interest in this subject it is referred to the publications mentioned therein, and especially to the one by Van Overschee and De Moor (1996).

The classical formulation of a MDOF system in continuous time as in Eq. (2.42) can be converted to a stochastic state space model in discrete time. The basic assumptions are that the physical structure under test behaves linearly and time-invariant and that it is subjected to multiple broad-banded random excitation. It is also assumed that the system response is discretely sampled with a sampling interval T so that the dynamical system can be formulated in discrete time as a stochastic state space system (Andersen and Brincker 2004; Aoki 1990; Ljung 1987; Peeters and De Roeck 1999; Söderström and Stoica 1989):

$$\boldsymbol{x}_{\mathrm{t+1}} = \boldsymbol{A}\boldsymbol{x}_{\mathrm{t}} + \boldsymbol{w}_{\mathrm{t}} \tag{4.1a}$$

$$\boldsymbol{y}_{\mathrm{t}} = \boldsymbol{D}\boldsymbol{x}_{\mathrm{t}} + \boldsymbol{v}_{\mathrm{t}} \tag{4.1b}$$

Eq. (4.1a) in the model structure is the state equation, which models the dynamic behaviour of the physical system, and Eq. (4.1b) is the observation equation, which models the output of the system. The measured response of the system, the output vector $\boldsymbol{y}_{\mathrm{t}}$, is generated by the two stochastic processes of the process noise $\boldsymbol{w}_{\mathrm{t}}$, which drives the system dynamics and the measurement noise $\boldsymbol{v}_{\mathrm{t}}$, the direct disturbance of the system response. The process and measurement noise are both assumed to be Gaussian white. The $n \times n$ state matrix $\boldsymbol{A}$ characterises the dynamics of the physical system and transforms the state of the system, the $n \times 1$ state vector $\boldsymbol{x}_{\mathrm{t}}$, to a new state $\boldsymbol{x}_{\mathrm{t+1}}$, given an $n \times 1$ input vector $\boldsymbol{w}_{\mathrm{t}}$. The dimension n of $\boldsymbol{x}_{\mathrm{t}}$ is the state space dimension. By forward multiplication of the $p \times n$ output matrix $\boldsymbol{D}$, the observable part of the dynamics is extracted from the state vector (Andersen and Brincker 2004).

The modal parameters damping, frequencies and shapes can be obtained from an eigenvalue decomposition of matrix $\boldsymbol{A}$:

$$\boldsymbol{A} = \boldsymbol{V}\left[\mu_{\mathrm{i}}\right]\boldsymbol{V}^{-1} \tag{4.2}$$

where $\boldsymbol{V}$ is the eigenvector matrix with the columns as the n eigenvectors of $\boldsymbol{A}$ and $[\mu_{\mathrm{i}}]$ is a diagonal matrix, which contains the associated eigenvalues of $\boldsymbol{A}$. For systems containing modes with damping below the critical one, which usually is the case for timber flooring structures, the eigenvalues and eigenvectors are organised in complex conjugate pairs with one pair for each mode. Then the number of modes in the system is $n/2$.

The natural frequency f_{i} and the damping ratio ζ_{i} can directly be determined from the eigenvalues for each of the $n/2$ modes as:

$$\begin{aligned}\lambda_{\mathrm{i}} &= \frac{\log(\mu_{\mathrm{i}})}{T}\\ f_{\mathrm{i}} &= \frac{|\lambda_{\mathrm{i}}|}{2\pi}\\ \zeta_{\mathrm{i}} &= -\frac{\mathrm{Re}(\lambda_{\mathrm{i}})}{|\lambda_{\mathrm{i}}|}\end{aligned} \tag{4.3}$$

where λ_{i} is the continuous time equivalent eigenvalue and $|\lambda_{\mathrm{i}}|$ is the absolute value of λ_{i}, and $\mathrm{Re}(\lambda_{\mathrm{i}})$ is the real part.

The mode shape vectors $\boldsymbol{\Phi}_{\mathrm{i}}$ of the $n/2$ modes are found from:

$$\boldsymbol{\Phi}_{\mathrm{i}} = \boldsymbol{D}\ \boldsymbol{V}_{\mathrm{i}} \tag{4.4}$$

where $\boldsymbol{V}_{\mathrm{i}}$ is the ith column of $\boldsymbol{V}$.

The SSI includes three identification classes: Unweighted Principal Components (UPC), Principal Components (PC) and Canonical Variate Analysis (CVA). Herlufsen et al. (2005) compared analysed data using all three methods and found them to be almost identical. CVA, however, required higher state space dimension compared to UPC or PC.

Redundant information in the measured responses can be reduced due to the selection of projection channels when processing the data, which saves computational time and can reduce the amount of noise modes considerably. The use of projection channels means the reduction of the number of channels that are used for the estimation. To identify the most useful channels, first the channel that correlates most with the other channels is sought as it is expected to contain maximum physical information. Automatically, all reference channels are selected as projection channels as they correlate most with the other channels due to their steady location and thus presence in all measurements. Other channels selected exhibit the least correlation with the first projection channels determined as they are expected to add the most new information. However, if a channel correlates insignificantly with any of the other channels, it will be disregarded (Andersen 2005). The use of projection channels does not affect the measured mode shape amplitudes at the different locations as the necessary information is kept (Peeters and De Roeck 1999).

Before processing the data, a maximum state space dimension needs to be specified, which depends on the number of modes in the model. The model does not only contain the physical but also noise modes, which needs to be considered. Furthermore, it should be mentioned again that the state space dimension is double the number of modes.

After processing the signals of one full test, a stabilisation diagram is shown with a number of modes for different state space dimensions for each measurement. The stabilisation diagram shows the natural frequencies of the estimated eigenvalues (Figure 4.20).

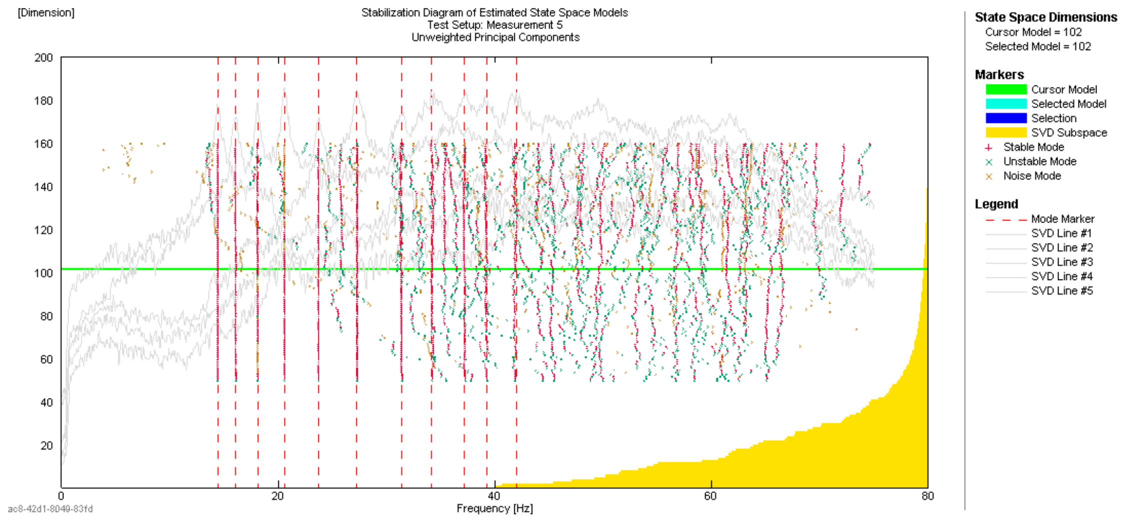


Figure 4.20: Stabilisation diagram of Measurement 5 of MWJ Floor A

The spectral densities in the background are included as wallpaper for orientation and validation purposes. Mode marker lines can be added to the spectral peaks to further aid locating the modes of interest. The singular values are illustrated as bars on the right of the figure and support the estimation of appropriate models. The selected subspace should include all singular values, which are significantly different from zero. If the model order is too low or too high, the amount of errors introduced in the model estimates is increased. This is illustrated in Figure 4.21.

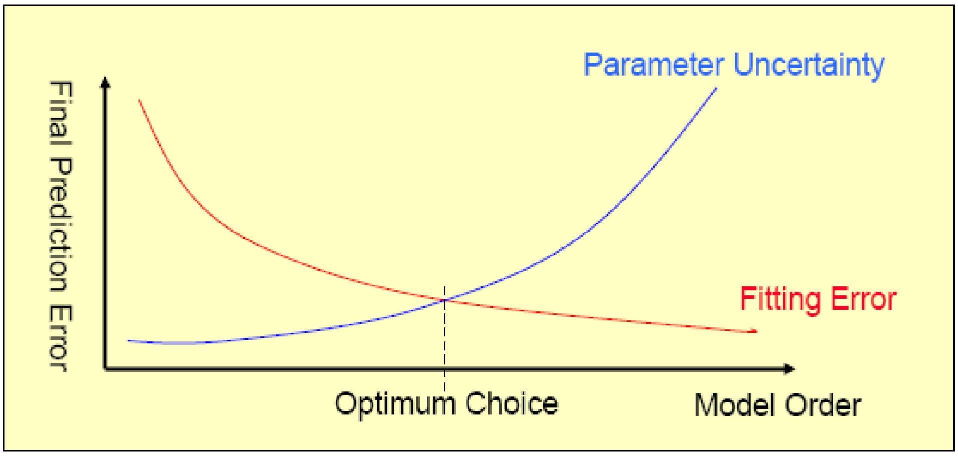


Figure 4.21: Choice of model order (Andersen 2005)

Physical modes in the stabilisation diagram are repeated for multiple model orders (Andersen 2005). To further clearly distinguish between physical and non-physical modes, stabilisation criteria are set or adjusted respectively for each measurement. This includes setting the maximum allowed deviation of natural frequencies, damping ratios, mode shape MAC and modal amplitude MAC between consecutive models. MAC stands for Modal Assurance Criterion and is a measure of consistency, not validity, between modal vectors (Harris and Piersol 2001). The deviation between consecutive models refers to the comparison of a model with the model of one order below. If the allowable deviations are exceeded, the corresponding mode is rated as unstable. Furthermore, boundaries can be set for damping ratios. All modes exhibiting damping ratios beyond this range are labelled as noise modes. Finally, a number of models with stable modes remain and a model that includes all the modes of interest needs to be selected for each measurement. The selected models are linked together to obtain the results for the full test.

The measured test data were sampled at 1500 Hz, resulting in a Nyquist frequency of 750 Hz. The signals were decimated by a factor of 10 and thus showing the spectral densities in a frequency range from 0 to 75 Hz. During the decimation an 8th order Chebyshev Type 1 low-pass filter is applied. From several analyses of some test data with respect to the EFDD, a number of 1024 frequency lines in the aforementioned frequency range were found to be appropriate, resulting in a frequency resolution of 0.07324 Hz. The output data from the test floors were optionally also high-pass filtered through a (Butterworth) filter of order 7 with a cut-off frequency selected between 7 and 15 Hz depending on the fundamental frequency determined for each system beforehand. The cut-off frequency was always chosen to be distant from the fundamental mode so as not to affect the actual modes. The filter was applied to enhance the stabilisation diagram in the SSI.

It was found that five projection channels contained sufficient amount of information and were therefore selected. For the SSI, it was then decided to use the UPC technique for the data presented in this chapter. The maximum state space dimension varied for different tests between a minimum of 80 and a maximum of 200. Examples of appropriate selection of state space dimensions and number of projection channels are given in the publication by Herlufsen et al. (2005). The maximum allowed deviation for stable modes between consecutive models was set to 0.2 Hz for the frequency and 0.2% for the damping ratio, and the default values of 0.05 and 0.5 kept for the mode shape MAC and modal amplitude MAC respectively. The boundaries for the damping ratio were set to 0.5% and 5.0%. Only for a few measurements of four flooring systems in the JJI test series, some of these strict criteria were slightly adjusted to obtain stable models including all modes of interest (damping deviation: 0.3% for one of five measurements for JJI 1 F, L and 2 S, and 0.4%, 0.5%, 0.6% for three of five measurements for JJI 2 P; maximum damping: 5.10% for one of five measurements for JJI 1 F).

5. Results of experimental investigations on timber I-joist floors

This section presents the results of the experimental investigations on the I-joist flooring systems, including natural frequencies and damping ratios of the principal modes of vibration and the static deflection under loading. Also the variation of the spacing between adjacent natural frequencies was addressed. Only the mode shapes with notable variations are presented. For the comparison of the mode shapes it should be noted that no absolute values for the amplitudes were obtained and the movement was relative. Sometimes, additional modes occurred in the tests, which may have been caused by imperfections in the structural supporting systems or some background noise. These modes could not be fully characterised and were not used in the analysis. Ohlsson (1982) categorised such modes, which could not be fully described, usually as false modes in his study (see also Chapter 3). Slight variation in natural frequencies and damping presented in this section may be found compared to those presented in earlier publications. This is due to either a reanalysis of the data, in which the signal processing was optimised or the uses of varied analysing methods like EFDD or SSI.

5.1 Results of TJI floor test series

Some of the results of this test series were published in Weckendorf et al. (2007). The deflection at the floor centre, the natural frequencies and corresponding damping ratios are summarised for the two test series in Table 5.1 and Table 5.2. As the results of the vibration tests are numerous, they are furthermore presented as comparisons in tabular format (Table 5.3 to Table 5.12). The upper half of these tables show the comparisons of the Test series 1 floors and the lower half those of Test series 2 floors as long as the series are not compared to each other. Concentration was paid to the first five modes of vibration. Modes which could not be fully identified were not included in the tables. This occurred for some of the higher Modes (1,4) and (1,5). It was slightly perplexing that the typical first order Mode (1,1), the mode with a single (half sine) wave in each plane direction, was not definitely detectable for any of the floors, which were supported along all four sides. This mode possesses no node in span or cross direction between the supports. As the whole floor surface was excited and an extensive number of well distributed measurement points used, the appropriate mode should had been set into motion and the response detected. The identified mode shapes of the same structure supported along two and along four sides are shown in Figure 5.4 of Section 5.1.1.1, using the example of Floors 2 I and J.

Table 5.1: Results of Test series TJI 1

Floor 1	w [mm]	$f_{(1,1)}$ [Hz]	$\zeta_{(1,1)}$ [%]	$f_{(1,2)}$ [Hz]	$\zeta_{(1,2)}$ [%]	$f_{(1,3)}$ [Hz]	$\zeta_{(1,3)}$ [%]	$f_{(1,4)}$ [Hz]	$\zeta_{(1,4)}$ [%]	$f_{(1,5)}$ [Hz]	$\zeta_{(1,5)}$ [%]
A	1.32	19.84	3.38	25.94	1.43	29.86	1.26	32.38	1.45	34.65	1.47
B	1.04	20.15	2.55	27.13	1.19	30.50	1.16	32.39	1.41	34.83	1.59
H	0.97	17.02	2.60	23.00	1.19	27.76	1.07	31.09	0.97	34.80	1.13
I	0.96	17.55	2.45	23.56	1.21	28.27	1.46	31.24	1.33	34.93	1.51
L	0.72	18.08	2.04	24.06	1.32	28.63	0.98	32.23	1.20	35.93	1.26
M	0.72	18.08	2.33	24.07	1.22	29.09	1.07	32.24	1.12	36.11	1.08
C	1.06	-	-	25.89	2.15	32.32	1.98	34.45	1.42	37.51	1.74
D	1.27	-	-	24.46	2.40	31.80	1.99	34.11	1.77	37.46	1.66
E	0.68	-	-	23.70	1.82	29.99	1.64	36.61	1.45	-	-
F	0.78	-	-	26.68	2.55	33.35	1.95	36.37	1.30	41.48	1.68
G	0.68	-	-	23.43	1.86	29.16	1.36	35.71	1.64	41.92	2.14
J	1.04	-	-	23.98	2.02	30.87	1.29	34.79	1.11	40.43	1.94
K	1.12	-	-	23.20	2.18	29.32	1.15	34.63	1.45	40.07	1.92
N	0.71	-	-	23.75	2.09	30.72	1.08	35.53	1.32	40.79	1.66
O	0.71	-	-	24.70	2.07	31.68	1.49	35.73	1.45	41.56	1.34

Table 5.2: Results of Test series TJI 2

Floor 2	w [mm]	$f_{(1,1)}$ [Hz]	$\zeta_{(1,1)}$ [%]	$f_{(1,2)}$ [Hz]	$\zeta_{(1,2)}$ [%]	$f_{(1,3)}$ [Hz]	$\zeta_{(1,3)}$ [%]	$f_{(1,4)}$ [Hz]	$\zeta_{(1,4)}$ [%]	$f_{(1,5)}$ [Hz]	$\zeta_{(1,5)}$ [%]
A	1.79	15.80	3.13	19.76	1.39	21.79	1.76	23.01	1.93	-	-
B	1.26	16.25	2.32	20.53	1.16	22.32	1.63	-	-	-	-
H	1.44	15.02	2.38	17.85	1.16	20.92	1.04	23.22	1.06	25.20	2.38
I	1.29	15.25	2.38	18.52	1.07	21.51	1.16	23.68	1.03	25.51	1.60
L	0.90	15.57	2.02	18.65	1.15	21.65	1.03	23.53	1.34	26.14	1.31
M	0.87	15.64	2.01	18.56	1.07	21.65	0.93	23.53	1.22	25.86	2.00
C	1.26	-	-	20.42	1.82	22.89	1.78	-	-	-	-
D	1.27	-	-	19.97	1.99	22.89	1.81	-	-	-	-
E	0.76	-	-	19.12	1.27	24.22	1.25	28.26	2.73	-	-
F	0.96	-	-	20.79	1.85	24.08	1.54	26.25	1.08	27.15	2.16
G	0.98	-	-	18.52	1.53	23.49	1.03	27.99	1.46	-	-
J	1.37	-	-	19.46	1.56	22.93	1.99	26.49	1.74	30.13	2.25
K	1.38	-	-	19.38	1.74	23.00	1.48	26.48	1.36	29.64	1.70
N	0.89	-	-	19.74	1.58	23.53	1.64	27.42	1.73	30.03	1.41
O	0.87	-	-	19.97	1.60	23.35	2.45	27.19	1.38	30.57	1.66

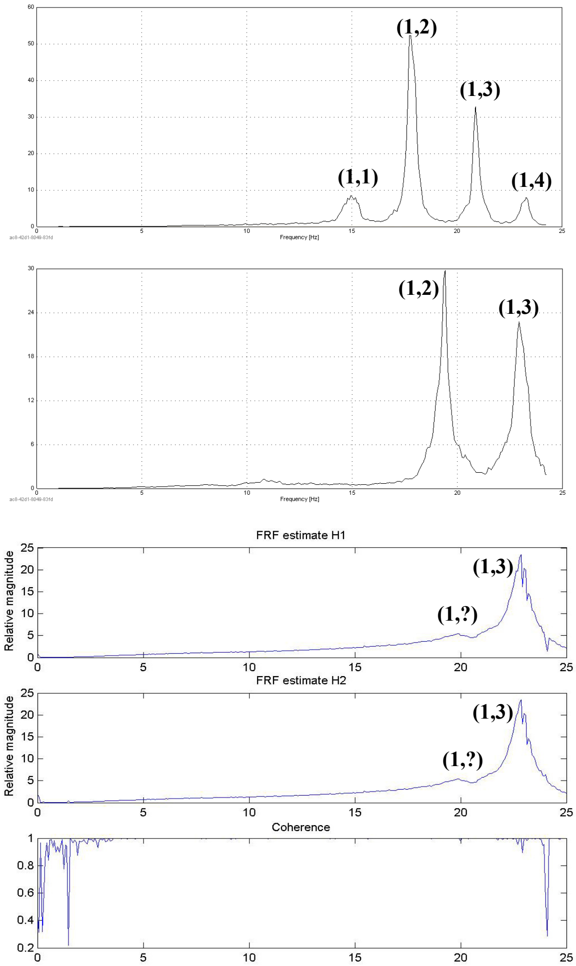
As some literature research showed, the phenomenon of non-detected Mode (1,1), rather seldom reported in the past, had been occurred in tests of other scientists. Kullaa and Talja (1999) found non-appearance of Mode (1,1) in the responses of two (out of three) light-weight steel-joist test floors, which were supported along four sides. An impact hammer was used for the excitation. A reason for the non-existence of Mode (1,1) was not given.

Pernica (1987) tested a flooring structure on site at two construction stages. The open-web steel joists were simply supported at the ends. The floor was excited by heel impacts. In some resulting frequency spectra, peaks for individual modes were not found. Pernica described the non-appearance of a peak by modal interference and the much larger response of the neighbouring mode, as the modes were closely spaced (e.g. Mode (1,1): 6.6 Hz, Mode (1,2): 7.0 Hz).

An example of the assumed vanished Mode (1,1) presented in Figure 5.1 illustrates first the EFDD frequency spectra of Floor 2 H (Figure 5.1(a)), which was supported at two edges and where Mode (1,1) was found, and then the spectra of Floor 2 K (Figure 5.1(b)), which was the same structure but supported at four edges where Mode (1,1) was not detectable. The frequency resolution used was rather high with a frequency line spacing of 0.07324 Hz.

The installation of the additional supports shifted the natural frequencies up as shown in Figure 5.1(a) and (b). Mode (1,1), however, appears not to be present with a peak in Figure 5.1(b). Also a small bump was often notable in a low frequency range of the four-side supported floors, which can be found at around 11 Hz for Floor 2 K. However, the frequency of Mode (1,1) for the two-side supported Floor 2 H was 15.02 Hz. The installation of the additional supports was not expected to result in a reduction of about 27% in frequency.

A possible explanation for the seemingly non-existence of Mode (1,1) could be found by studying the equation to calculate natural frequencies of rectangular orthotropic plates simply supported along all four edges (Eq. (3.6)). This equation was used to calculate the first five first-order modes with the properties of Floor 2 K but increasing floor width from 2.0 to 5.0 m gradually at an increment of 0.5 m. Composite action was not considered in the calculations. The results are presented in Figure 5.2.

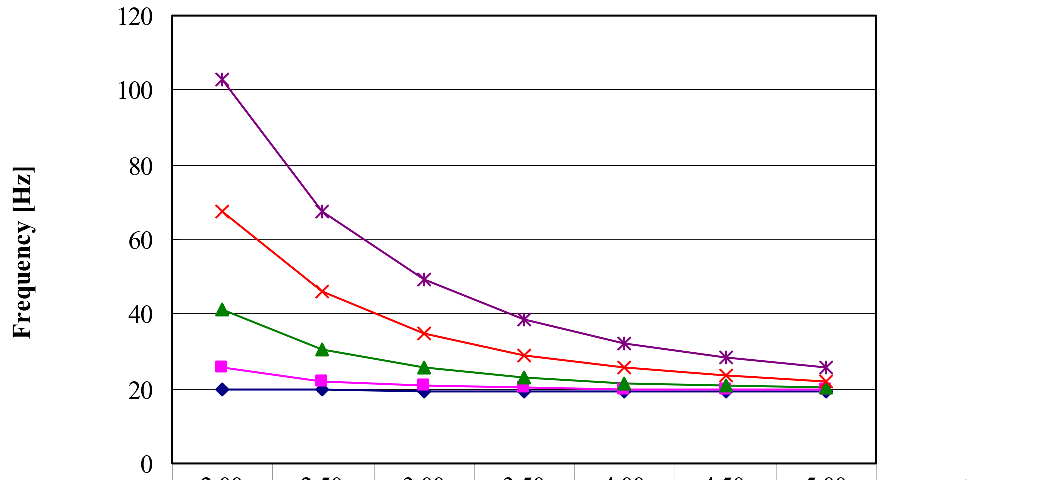


(a) Response of Floor 2 A, supported along two sides, in frequency spectrum determined from output-only modal analysis

(b) Response of Floor 2 K, same floor as above but supported along four sides, in frequency spectrum determined from output-only modal analysis

(c) Response of Floor 2 K in frequency spectra due to impact hammer test, and coherence function confirming validity of test

Figure 5.1: Illustration of the "disappearance" of Mode (1,1)



	2.00	2.50	3.00	3.50	4.00	4.50	5.00
Mode (1,1)	19.92	19.64	19.52	19.47	19.44	19.43	19.42
Mode (1,2)	25.56	22.23	20.87	20.24	19.92	19.74	19.64
Mode (1,3)	41.50	30.63	25.56	23.01	21.65	20.87	20.41
Mode (1,4)	67.59	45.96	34.99	29.00	25.56	23.50	22.23
Mode (1,5)	102.66	67.59	49.14	38.57	32.17	28.15	25.56

Figure 5.2: Variation in natural frequencies for increase in width of TJI Floor 2 K

As is shown, the natural frequencies for each mode decreased with increasing floor width and the spacing between the natural frequencies was getting closer. For relatively large width, especially the frequencies of Modes (1,1) and (1,2) almost coincide with each other. The calculated frequencies of the higher modes show that the prediction is not always matching the test results. However, they could give an indication for which reason the Mode (1,1) of four-side supported floors could not be identified. This is close to the reason given by Pernica (1987).

To double-check the results obtained for Floor 2 K, a single sensor was mounted at the centre of the floor where the highest amplitude for Mode (1,1) would be expected. The floor was then excited with an Endevco impact hammer (Model 2302 from PCB Piezotronics) close to the sensor. The input and output were measured whereas the impact was repeated four times (corresponding to five impacts in total) for one measurement to obtain an averaged result. A Matlab programme, written by Dr. Palle Andersen, was used for the analysis. This testing method was selected as contrast to the method used regularly, to possibly exclude the latter as the cause of the phenomenon.

The central line in longitudinal direction is a node for Mode (1,2). As the response was intentionally measured at the floor centre while the floor was excited close to it, response of Mode (1,2) may theoretically not be present in the frequency spectrum. Imperfections in the flooring structure or placing the sensor not exactly on the node respectively could lead to some detectable response of this mode.

Therefore, the first peak in these frequency spectra could correspond to Mode (1,2), to rather low response of Mode (1,1) or to a mixture of both if they were so closely spaced (Figure 5.1(c)). Mode (1,3) was well represented. The frequency spectrum before Mode (1,2) was rather flat and a bump was not visible for this single measurement. The coherence function ("*For each frequency*" the coherence function "*shows the degree of linear relationship between the measured input and output signals*" (Døssing 1988a)) was close or equal to unity in the critical area, which confirmed the validity of this test. The peaks from this testing method matched those of the other testing method.

If Modes (1,1) and (1,2) are located at about the same frequency but only the shape of Mode (1,2) can be identified, e.g. without any observable interference of the shape corresponding to Mode (1,1), Mode (1,2) is dominating and becomes fundamental. This is supported by the findings from Kullaa and Talja (1999), who measured dynamic displacements on their test structures. These displacements were usually higher at the quarter-points in transverse direction, the anti-nodes of Mode (1,2), than at the centre of the floor.

In an earlier publication (Weckendorf et al. (2006)), Mode (1,2) was already considered to be the fundamental mode of the four-side supported test floors where Mode (1,1) could not be definitely identified. Nevertheless, the described bump was referred to as the first bending mode since for some test floors bending could be found in the corresponding mode shapes. This description for the bump is not maintained in this thesis as it was finally considered not to represent the actual Mode (1,1). The frequencies of Mode (1,1) of the floors supported along all four sides are not presented in the tables of this chapter as they could not be definitely identified.

The net deflections at floor centre are presented in column charts in the following subsections for easy comparisons. The results of Test series 1 and 2 are presented in different figures as long as they are not compared to each other. For the latter they are divided into figures for two- and four-side supported floors. The measurement at mid-span of the edges in span direction showed zero movements or little upward movements when loading the floor. This indicated that only joists closer to the floor centre deflected notably and that the load sharing effect was limited. The load sharing effect was not further investigated for this test series.

5.1.1 Support conditions

5.1.1.1 Supporting the floors along two and four sides

Table 5.3 and Table 5.4 show the natural frequencies and damping ratios respectively, and their degree of variation for the compared flooring structures. It can be noted that almost all frequencies were increasing if the supports under the outer floor joists were installed. The efficiency was mainly raised when stiffer flooring materials were used, such as stiffer I-joists for the larger floors or stiffer deck, as can in particular be noted from the effect on Mode (1,2). For the Test series 1 floors, the frequencies of this mode decreased for the floors decked with OSB and were marginally affected for the floors decked with particleboard. For the Test series 2 floors, the frequencies of the same mode were marginally affected for the floors decked with OSB and increased for the floors decked with particleboard. The efficiency was generally increasing for successive modes, except Mode (1,4) comparing Floors 1 A and D, leading to a wider separation of the natural frequencies. An increase in the natural frequencies can generally be expected due to raised stiffness from the addition of the supports under the outer floor edges.

Table 5.3: Natural frequencies of two- and four-side supported floors

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	19.84	25.94	29.86	32.38	34.65
1 D	-	24.46	31.80	34.11	37.46
Variation[%]		**- 5.71**	**+ 6.50**	**+ 5.34**	**+ 8.11**
1 B	20.15	27.13	30.50	32.39	34.83
1 C	-	25.89	32.32	34.45	37.51
Variation [%]		**- 4.57**	**+ 5.97**	**+ 6.36**	**+ 7.69**
1 H	17.02	23.00	27.76	31.09	34.80
1 K	-	23.20	29.32	34.63	40.07
Variation[%]		**+ 0.87**	**+ 5.62**	**+ 11.39**	**+ 15.14**
1 I	17.55	23.56	28.27	31.24	34.93
1 J	-	23.98	30.87	34.79	40.43
Variation [%]		**+ 1.78**	**+ 9.20**	**+ 11.36**	**+ 15.75**
1 L	18.08	24.06	28.63	32.23	35.93
1 N	-	23.75	30.72	35.53	40.79
Variation[%]		**- 1.29**	**+ 7.30**	**+ 10.24**	**+ 13.53**
1 M	18.08	24.07	29.09	32.24	36.11
1 O	-	24.70	31.68	35.73	41.56
Variation [%]		**+ 2.62**	**+ 8.90**	**+ 10.83**	**+ 15.09**
2 A	15.80	19.76	21.79	23.01	-
2 D	-	19.97	22.89	-	-
Variation[%]		**+ 1.06**	**+ 5.05**		
2 B	16.25	20.53	22.32	-	-
2 C	-	20.42	22.89	-	-
Variation [%]		**- 0.54**	**+ 2.55**		
2 H	15.02	17.85	20.92	23.22	25.20
2 K	-	19.38	23.00	26.48	29.64
Variation[%]		**+ 8.57**	**+ 9.94**	**+ 14.04**	**+ 17.62**
2 I	15.25	18.52	21.51	23.68	25.51
2 J	-	19.46	22.93	26.49	30.13
Variation [%]		**+ 5.08**	**+ 6.60**	**+ 11.87**	**+ 18.11**
2 L	15.57	18.65	21.65	23.53	26.14
2 N	-	19.74	23.53	27.42	30.03
Variation[%]		**+ 5.84**	**+ 8.68**	**+ 16.53**	**+ 14.88**
2 M	15.64	18.56	21.65	23.53	25.86
2 O	-	19.97	23.35	27.19	30.57
Variation [%]		**+ 7.60**	**+ 7.85**	**+ 15.55**	**+ 18.21**

Table 5.4: Damping ratios of two- and four-side supported floors

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 A	3.38	1.43	1.26	1.45	1.47
1 D	-	2.40	1.99	1.77	1.66
Variation[%]		**+ 67.83**	**+ 57.94**	**+ 22.07**	**+ 12.93**
1 B	2.55	1.19	1.16	1.41	1.59
1 C	-	2.15	1.98	1.42	1.74
Variation [%]		**+ 80.67**	**+ 70.69**	**+ 0.71**	**+ 9.43**
1 H	2.60	1.19	1.07	0.97	1.13
1 K	-	2.18	1.15	1.45	1.92
Variation[%]		**+ 83.19**	**+ 7.48**	**+ 49.48**	**+ 69.91**
1 I	2.45	1.21	1.46	1.33	1.51
1 J	-	2.02	1.29	1.11	1.94
Variation [%]		**+ 66.94**	**- 11.64**	**- 16.54**	**+ 28.48**
1 L	2.04	1.32	0.98	1.20	1.26
1 N	-	2.09	1.08	1.32	1.66
Variation[%]		**+ 58.33**	**+ 10.20**	**+ 10.00**	**+ 31.75**
1 M	2.33	1.22	1.07	1.12	1.08
1 O	-	2.07	1.49	1.45	1.34
Variation [%]		**+ 69.67**	**+ 39.25**	**+ 29.46**	**+ 24.07**
2 A	3.13	1.39	1.76	1.93	-
2 D	-	1.99	1.81	-	-
Variation[%]		**+ 43.17**	**+ 2.84**		
2 B	2.32	1.16	1.63	-	-
2 C	-	1.82	1.78	-	-
Variation [%]		**+ 56.90**	**+ 9.20**		
2 H	2.38	1.16	1.04	1.06	2.38
2 K	-	1.74	1.48	1.36	1.70
Variation[%]		**+ 50.00**	**+ 42.31**	**+ 28.30**	**- 28.57**
2 I	2.38	1.07	1.16	1.03	1.60
2 J	-	1.56	1.99	1.74	2.25
Variation [%]		**+ 45.79**	**+ 71.55**	**+ 68.93**	**+ 40.63**
2 L	2.02	1.15	1.03	1.34	1.31
2 N	-	1.58	1.64	1.73	1.41
Variation[%]		**+ 37.39**	**+ 59.22**	**+ 29.10**	**+ 7.63**
2 M	2.01	1.07	0.93	1.22	2.00
2 O	-	1.60	2.45	1.38	1.66
Variation [%]		**+ 49.53**	**+ 163.44**	**+ 13.11**	**- 17.00**

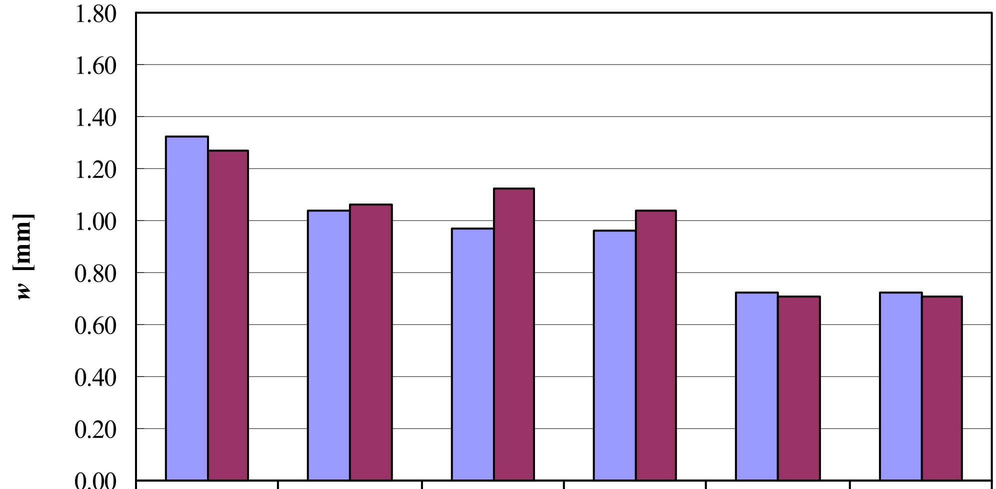
The comparison of damping ratios (Table 5.4) showed a clear trend with highly raised damping after introducing the additional floor supports. The damping of Mode (1,2) increased by at least 37.15% and up to 82.24% with 59.12% on average. Also for the damping ratios of the other detected modes, there were mainly relative growths in the range of double percentage figures. Three comparisons show a decrease in damping. This is a relative small amount of dissent, considering the total amount of results, if damping ratios are compared. The increase in damping can be explained by the additional friction between floor joists and added supports.

The comparisons of the net deflections at the floor centre are shown in Figure 5.3. The variation was very small, with a maximum of 0.15 mm apart from the comparison of Floors 2 A and D where the deflection of Floor 2 A showed rather odd high deflection. There was rather no effect on the floors whose deck was fixed to the joists with adhesives and screws. The general low variation in deflection results from the limited load-sharing effects to neighbouring joists and limited end fixity from added supports, no matter if the floor was simply supported or screw-fixed to the supports.

The mode shapes varied depending on whether the floors were two- or four-side supported as illustrated for Floors 2 I and 2 J in Figure 5.4. As only first order modes were considered, there was always only one (half sine) wave (up or down if not a node) in longitudinal direction. The varying shapes in transverse direction are therefore presented.

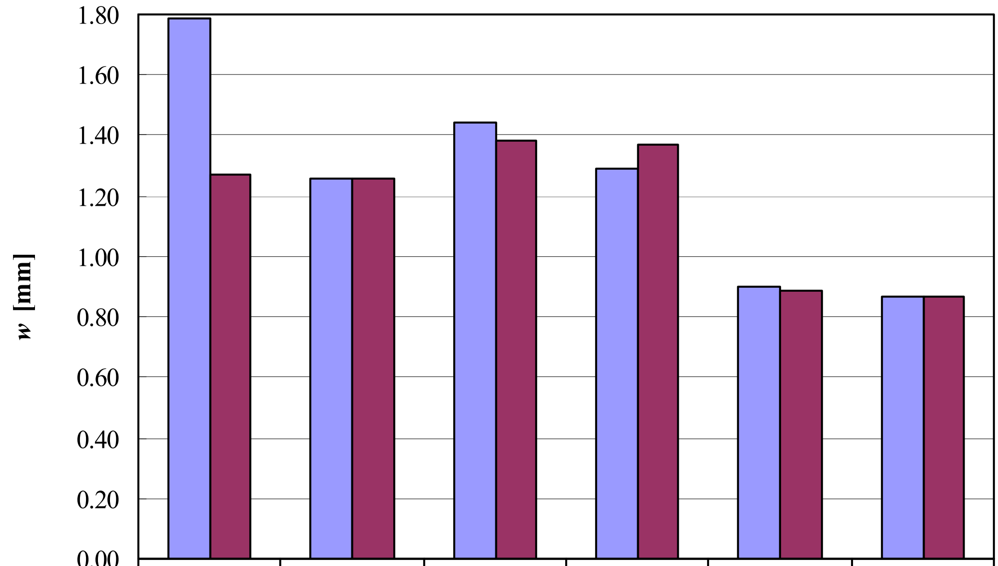
It can be noted that there was always an anti-node in the floor centre for the odd mode numbers (e.g. Modes (1,1), (1,3) and (1,5)) whereas the same location was a node for the even mode numbers (e.g. Modes (1,2) and (1,4)). The location of possible other nodes and anti-nodes depended on whether the floor was supported at two or four sides. There was always considerable movement at unsupported edges (Figure 5.4(a)) whereas there was little of it at supported sides (Figure 5.4(b)).

As mentioned earlier, Mode (1,1) was not found for four-side supported floors. The fundamental mode for these floors should thus be Mode (1,2), which had anti-nodes at about the quarter points of the floor width. Mode (1,3) was found with anti-nodes in transverse direction at the centre of the floor and around the sixth points from each edge. To clearly locate the anti-nodes of higher modes, the grid used for the measurement needed to be smaller meshed. For two-side supported floors, Mode (1,1) was found, which had highest movement at the centre of the floor. Mode (1,2) and Mode (1,3) showed relatively high movement at the unsupported edges. The latter exhibited also large movement at the centre of the floor. The mesh was again not sufficiently fine to clearly locate the anti-nodes of higher modes.



Floor 1	A - D	B - C	H - K	I - J	L - N	M - O
2-side supported	1.32	1.04	0.97	0.96	0.72	0.72
4-side supported	1.27	1.06	1.12	1.04	0.71	0.71

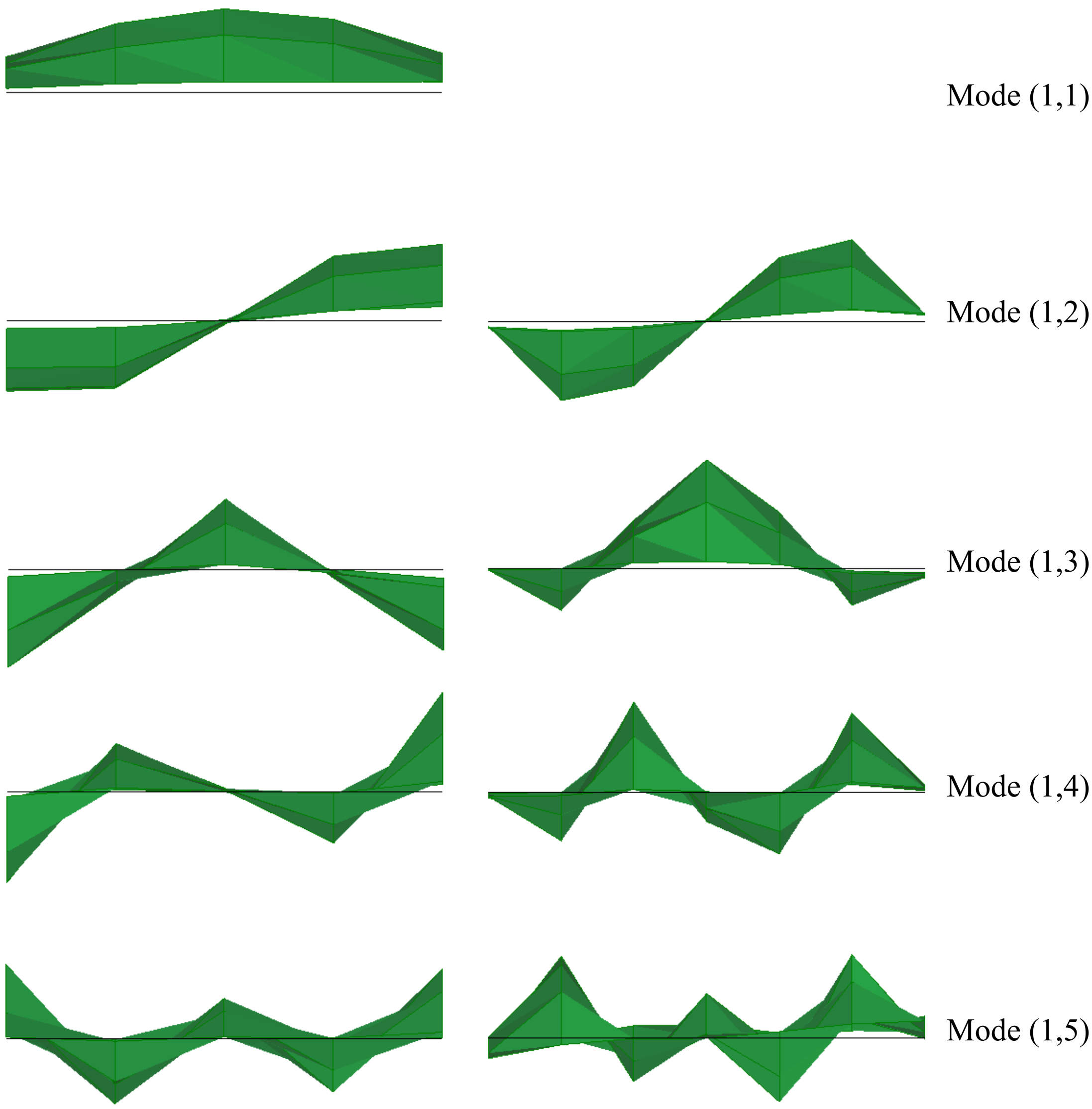
(a) Test series 1



Floor 2	A - D	B - C	H - K	I - J	L - N	M - O
2-side supported	1.79	1.26	1.44	1.29	0.90	0.87
4-side supported	1.27	1.26	1.38	1.37	0.89	0.87

(b) Test series 2

Figure 5.3: Deflections at the centre of the two- and four-side supported floors



(a) Floor 2 I (two-side supported) (b) Floor 2 J (four-side supported)

Figure 5.4: Mode shapes of the same structure two- and four-side supported

5.1.1.2 Raising the support end fixity

Table 5.5 and Table 5.6 present the comparisons of natural frequencies and damping ratios for the floors with increased support end fixity. The frequencies of the floors with bonding between deck and joists, which were supported at two edges, were little affected. A similar low effect existed for all four-side supported floors of Test series 2. A slight shift to higher frequencies was found otherwise with an impact in particular on Modes (1,1) to (1,3).

The damping ratios mainly decreased for Modes (1,1) and (1,2). For higher modes the variation of damping was rather inconclusive, but the number of decreasing damping ratios prevailed.

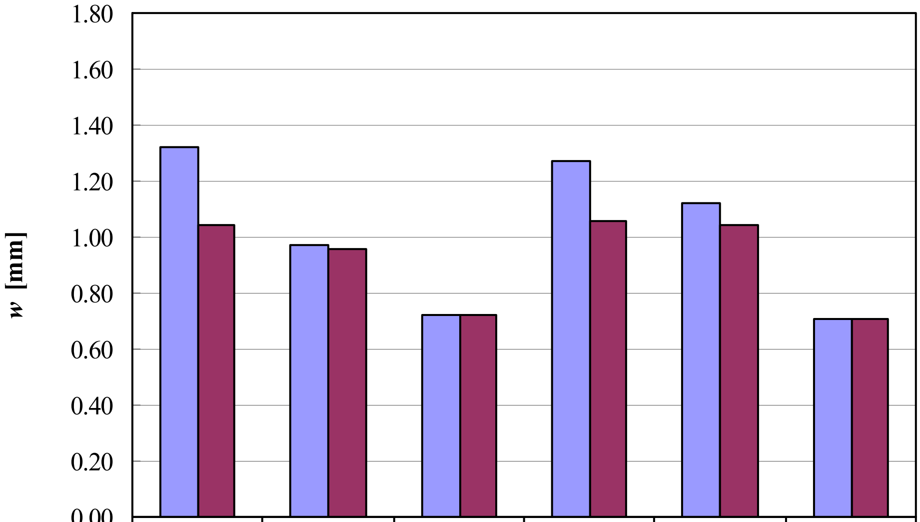
The method used to fix the floors to the supports resulted in a decrease of the deflection at the floor centre or was little influential (Figure 5.5). The floors decked with a 15 mm thick OSB layer all had certain decrease in deflection with at least 16.54% for three comparisons. For the deck fixed to the joists by adhesives in addition to screws, the effect was little.

Table 5.5: Natural frequencies of floors simply supported and screw-fixed to supports

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	19.84	25.94	29.86	32.38	34.65
1 B	20.15	27.13	30.50	32.39	34.83
Variation[%]	**+ 1.56**	**+ 4.59**	**+ 2.14**	**+ 0.03**	**+ 0.52**
1 H	17.02	23.00	27.76	31.09	34.80
1 I	17.55	23.56	28.27	31.24	34.93
Variation[%]	**+ 3.11**	**+ 2.43**	**+ 1.84**	**+ 0.48**	**+ 0.37**
1 L	18.08	24.06	28.63	32.23	35.93
1 M	18.08	24.07	29.09	32.24	36.11
Variation[%]	**0.00**	**+ 0.04**	**+ 1.61**	**+ 0.03**	**+ 0.50**
1 D	-	24.46	31.80	34.11	37.46
1 C	-	25.89	32.32	34.45	37.51
Variation [%]		**+ 5.85**	**+ 1.64**	**+ 1.00**	**+ 0.13**
1 K	-	23.20	29.32	34.63	40.07
1 J	-	23.98	30.87	34.79	40.43
Variation [%]		**+ 3.36**	**+ 5.29**	**+ 0.46**	**+ 0.90**
1 N	-	23.75	30.72	35.53	40.79
1 O	-	24.70	31.68	35.73	41.56
Variation [%]		**+ 4.00**	**+ 3.13**	**+ 0.56**	**+ 1.89**
2 A	15.80	19.76	21.79	23.01	-
2 B	16.25	20.53	22.32	-	-
Variation[%]	**+ 2.85**	**+ 3.90**	**+ 2.43**		
2 H	15.02	17.85	20.92	23.22	25.20
2 I	15.25	18.52	21.51	23.68	25.51
Variation[%]	**+ 1.53**	**+ 3.75**	**+ 2.82**	**+ 1.98**	**+ 1.23**
2 L	15.57	18.65	21.65	23.53	26.14
2 M	15.64	18.56	21.65	23.53	25.86
Variation[%]	**+ 0.45**	**- 0.48**	**0.00**	**0.00**	**- 1.07**
2 D	-	19.97	22.89	-	-
2 C	-	20.42	22.89	-	-
Variation [%]		**+ 2.25**	**0.00**		
2 K	-	19.38	23.00	26.48	29.64
2 J	-	19.46	22.93	26.49	30.13
Variation [%]		**+ 0.41**	**- 0.30**	**+ 0.04**	**+ 1.65**
2 N	-	19.74	23.53	27.42	30.03
2 O	-	19.97	23.35	27.19	30.57
Variation [%]		**+ 1.17**	**- 0.76**	**- 0.84**	**+ 1.80**

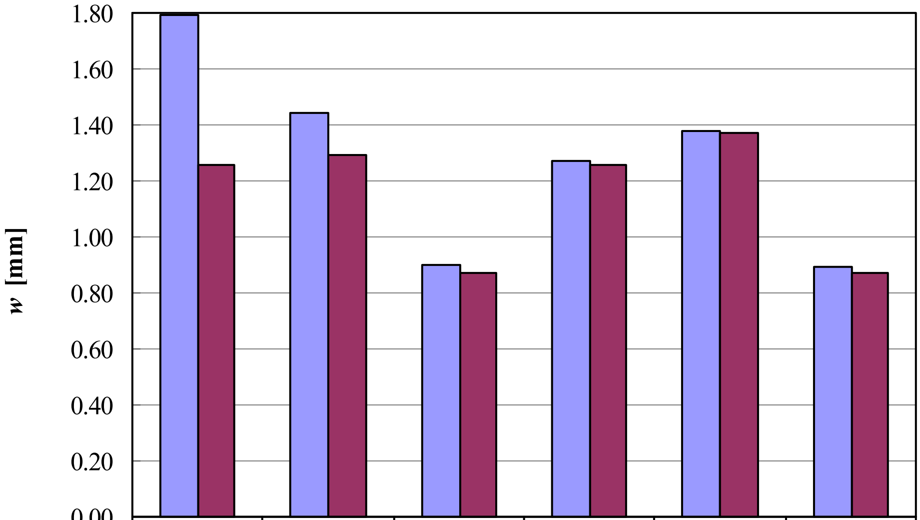
Table 5.6: Damping ratios of floors simply supported and screw-fixed to supports

Floor	$\zeta_{(1,1)}$ **[%]**	$\zeta_{(1,2)}$ **[%]**	$\zeta_{(1,3)}$ **[%]**	$\zeta_{(1,4)}$ **[%]**	$\zeta_{(1,5)}$ **[%]**
1 A	3.38	1.43	1.26	1.45	1.47
1 B	2.55	1.19	1.16	1.41	1.59
Variation[%]	**- 24.56**	**- 16.78**	**- 7.94**	**- 2.76**	**+ 8.16**
1 H	2.60	1.19	1.07	0.97	1.13
1 I	2.45	1.21	1.46	1.33	1.51
Variation[%]	**- 5.77**	**+ 1.68**	**+ 36.45**	**+ 37.11**	**+ 33.63**
1 L	2.04	1.32	0.98	1.20	1.26
1 M	2.33	1.22	1.07	1.12	1.08
Variation[%]	**+ 14.22**	**- 7.58**	**+ 9.18**	**- 6.67**	**- 14.29**
1 D	-	2.40	1.99	1.77	1.66
1 C	-	2.15	1.98	1.42	1.74
Variation [%]		**- 10.42**	**- 0.50**	**- 19.77**	**+ 4.82**
1 K	-	2.18	1.15	1.45	1.92
1 J	-	2.02	1.29	1.11	1.94
Variation [%]		**- 7.34**	**+ 12.17**	**- 23.45**	**+ 1.04**
1 N	-	2.09	1.08	1.32	1.66
1 O	-	2.07	1.49	1.45	1.34
Variation [%]		**- 0.96**	**+ 37.96**	**+ 9.85**	**- 19.28**
2 A	3.13	1.39	1.76	1.93	-
2 B	2.32	1.16	1.63	-	-
Variation[%]	**- 25.88**	**- 16.55**	**- 7.39**		
2 H	2.38	1.16	1.04	1.06	2.38
2 I	2.38	1.07	1.16	1.03	1.60
Variation[%]	**0.00**	**- 7.76**	**+ 11.54**	**- 2.83**	**- 32.77**
2 L	2.02	1.15	1.03	1.34	1.31
2 M	2.01	1.07	0.93	1.22	2.00
Variation[%]	**- 0.50**	**- 6.96**	**- 9.71**	**- 8.96**	**+ 52.67**
2 D	-	1.99	1.81	-	-
2 C	-	1.82	1.78	-	-
Variation [%]		**- 8.54**	**- 1.66**		
2 K	-	1.74	1.48	1.36	1.70
2 J	-	1.56	1.99	1.74	2.25
Variation [%]		**- 10.34**	**+ 34.46**	**+ 27.94**	**+ 32.35**
2 N	-	1.58	1.64	1.73	1.41
2 O	-	1.60	2.45	1.38	1.66
Variation [%]		**+ 1.27**	**+ 49.39**	**- 20.23**	**+ 17.73**



Floor 1	A - B	H - I	L - M	D - C	K - J	N - O
Simply supported	1.32	0.97	0.72	1.27	1.12	0.71
Screw-fixed to supports	1.04	0.96	0.72	1.06	1.04	0.71

(a) Test series 1



Floor 2	A - B	H - I	L - M	D - C	K - J	N - O
Simply supported	1.79	1.44	0.90	1.27	1.38	0.89
Screw-fixed to supports	1.26	1.29	0.87	1.26	1.37	0.87

(b) Test series 2

Figure 5.5: Deflections at the centre of floors simply supported and screw-fixed to the supports

5.1.2 Effect of decking types

Table 5.7 and Table 5.8 compare the frequencies and damping respectively for the floors with different decking types. Using the stiffer and heavier particleboard on top of the less stiff and less heavy OSB or instead of it, resulted in reduced frequencies of Modes (1,1) and (1,2) and raised frequencies for higher modes with little effect on frequencies in-between. The turning point was varying. The different effects on frequencies of lower modes and higher modes respectively are caused by varying influences of mass and stiffness. This is further discussed in Section 5.1.5. For the larger floors, the reduction for the lower modes was less strong with an average of 5.55% for the frequency corresponding to Mode (1,1) compared to an average reduction of 13.56% for the shorter floors, and the rise at the higher modes was stronger.

Table 5.7: Natural frequencies of floors with varied decking

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	19.84	25.94	29.86	32.38	34.65
1 H	17.02	23.00	27.76	31.09	34.80
Variation[%]	**- 14.21**	**- 11.33**	**- 7.03**	**- 3.98**	**+ 0.43**
1 B	20.15	27.13	30.50	32.39	34.83
1 I	17.55	23.56	28.27	31.24	34.93
Variation[%]	**- 12.90**	**- 13.16**	**- 7.31**	**- 3.55**	**+ 0.29**
1 C	-	25.89	32.32	34.45	37.51
1 J	-	23.98	30.87	34.79	40.43
Variation [%]		**- 7.38**	**- 4.49**	**+ 0.99**	**+ 7.78**
1 E	-	23.70	29.99	36.61	-
Variation [%]		**- 8.46**	**- 7.21**	**+ 6.27**	
1 D	-	24.46	31.80	34.11	37.46
1 K	-	23.20	29.32	34.63	40.07
Variation [%]		**- 5.15**	**- 7.80**	**+ 1.52**	**+ 6.97**
1 F	-	26.68	33.35	36.37	41.48
1 O	-	24.70	31.68	35.73	41.56
Variation [%]		**- 7.42**	**- 5.01**	**- 1.76**	**+ 0.19**
1 G	-	23.43	29.16	35.71	41.92
Variation [%]		**- 12.18**	**- 12.56**	**- 1.81**	**+ 1.06**
2 A	15.80	19.76	21.79	23.01	-
2 H	15.02	17.85	20.92	23.22	25.20
Variation[%]	**- 4.94**	**- 9.67**	**- 3.99**	**+ 0.91**	
2 B	16.25	20.53	22.32	-	-
2 I	15.25	18.52	21.51	23.68	25.51
Variation[%]	**- 6.15**	**- 9.79**	**- 3.63**		
2 C	-	20.42	22.89	-	-
2 J	-	19.46	22.93	26.49	30.13
Variation [%]		**- 4.70**	**+ 0.17**		
2 E	-	19.12	24.22	28.26	-
Variation [%]		**- 6.37**	**+ 5.81**		
2 D	-	19.97	22.89	-	-
2 K	-	19.38	23.00	26.48	29.64
Variation [%]		**- 2.95**	**+ 0.48**		
2 F	-	20.79	24.08	26.25	27.15
2 O	-	19.97	23.35	27.19	30.57
Variation [%]		**- 3.94**	**- 3.03**	**+ 3.58**	**+ 12.60**
2 G	-	18.52	23.49	27.99	-
Variation [%]		**- 10.92**	**- 2.45**	**+ 6.63**	

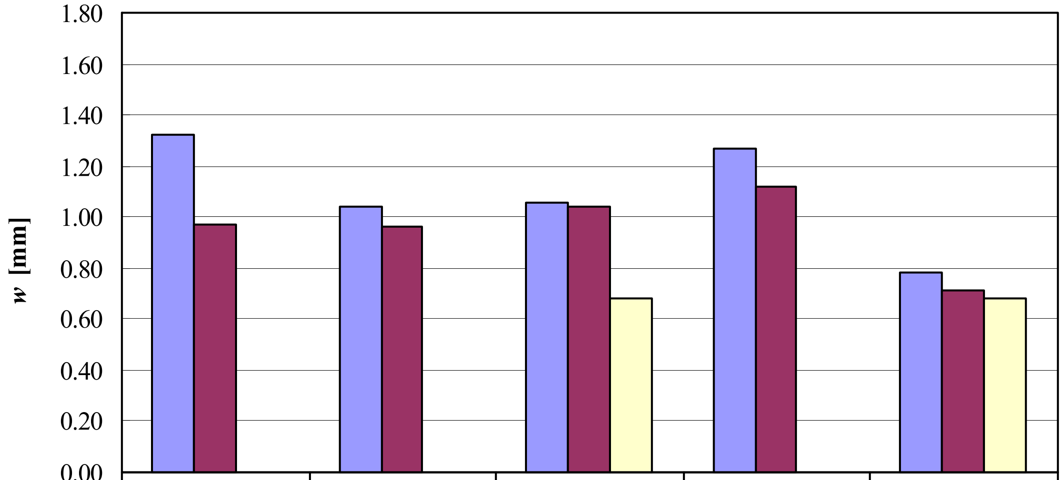
Table 5.8: Damping ratios of floors with varied decking

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 A	3.38	1.43	1.26	1.45	1.47
1 H	2.60	1.19	1.07	0.97	1.13
Variation[%]	**- 23.08**	**- 16.78**	**- 15.08**	**- 33.10**	**- 23.13**
1 B	2.55	1.19	1.16	1.41	1.59
1 I	2.45	1.21	1.46	1.33	1.51
Variation[%]	**- 3.92**	**+ 1.68**	**+ 25.86**	**- 5.67**	**- 5.03**
1 C	-	2.15	1.98	1.42	1.74
1 J	-	2.02	1.29	1.11	1.94
Variation [%]		**- 6.05**	**- 34.85**	**- 21.83**	**+ 11.49**
1 E	-	1.82	1.64	1.45	-
Variation [%]		**- 15.35**	**- 17.17**	**+ 2.11**	
1 D	-	2.40	1.99	1.77	1.66
1 K	-	2.18	1.15	1.45	1.92
Variation [%]		**- 9.17**	**- 42.21**	**- 18.08**	**+ 15.66**
1 F	-	2.55	1.95	1.30	1.68
1 O	-	2.07	1.49	1.45	1.34
Variation [%]		**- 18.82**	**- 23.59**	**+ 11.54**	**- 20.24**
1 G	-	1.86	1.36	1.64	2.14
Variation [%]		**- 27.06**	**- 30.26**	**+ 26.15**	**+ 27.38**
2 A	3.13	1.39	1.76	1.93	-
2 H	2.38	1.16	1.04	1.06	2.38
Variation[%]	**- 23.96**	**- 16.55**	**- 40.91**	**- 45.08**	
2 B	2.32	1.16	1.63	-	-
2 I	2.38	1.07	1.16	1.03	1.60
Variation[%]	**+ 2.59**	**- 7.76**	**- 28.83**		
2 C	-	1.82	1.78	-	-
2 J	-	1.56	1.99	1.74	2.25
Variation [%]		**- 14.29**	**+ 11.80**		
2 E	-	1.27	1.25	2.73	-
Variation [%]		**- 30.22**	**- 29.78**		
2 D	-	1.99	1.81	-	-
2 K	-	1.74	1.48	1.36	1.70
Variation [%]		**- 12.56**	**- 18.23**		
2 F	-	1.85	1.54	1.08	2.16
2 O	-	1.60	2.45	1.38	1.66
Variation [%]		**- 13.51**	**+ 59.09**	**+ 27.78**	**- 23.15**
2 G	-	1.53	1.03	1.46	-
Variation [%]		**- 17.30**	**- 33.12**	**+ 35.19**	

For the two-side supported floors, the frequencies of the Modes (1,1) and (1,2) became closer spaced whereas higher frequencies became more separated. For the four-side supported floors of Test series 2, the separation of frequencies was generally widened. For those of Test series 1, the frequency spacing varied differently between Modes (1,2) and (1,3) whereas the one between the higher adjacent modes was always raised.

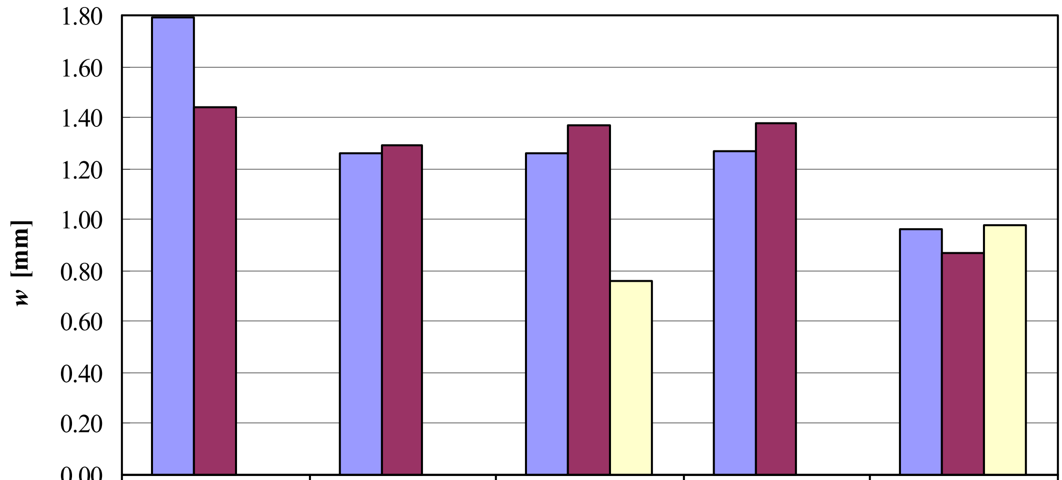
The damping ratios tended to decrease in a predominant majority especially for the Modes (1,1) to (1,3) and especially for the floors with double layer deck (see also Section 5.1.5 and Chapter 9).

Figure 5.6 illustrates the deflection at the floor centre for varying floor deck. The figures regarding Floor 1 show a clear trend with decreasing deflections for increasing stiffness, which could be expected.



Floor 1	A - H	B - I	C - J - E	D - K	F - O - G
OSB	1.32	1.04	1.06	1.27	0.78
P5	0.97	0.96	1.04	1.12	0.71
OSB + P5			0.68		0.68

(a) Test series 1



Floor 2	A - H	B - I	C - J - E	D - K	F - O - G
OSB	1.79	1.26	1.26	1.27	0.96
P5	1.44	1.29	1.37	1.38	0.87
OSB + P5			0.76		0.98

(b) Test series 2

Figure 5.6: Deflections at the floor centre of floors with varied decking

The figures regarding Floor 2 are not in full agreement with this trend. This could be explained by using the different samples needed, whereas one sample was always decked with OSB (+ particleboard) and the other sample was always decked with particleboard as detailed in Section 4.1. Therefore, some stiffness variation in the joist material could have caused these results. This trend was confirmed by the variation in frequency as described above where the reduction in frequency of lower modes was less strong and the increase in frequency for higher modes was stronger for the larger floors as compared to the smaller ones. If no adhesives were used, the double layer deck led to a significant decrease in deflection of 35.85% and 39.68% respectively compared to the floors decked with a single 15 mm thick OSB layer. When adhesive was used in addition to screws to fix the deck to the joists, the absolute variation in deflection was very small.

5.1.3 Effect of deck-to-joist fixing method

Table 5.9 and Table 5.10 present the natural frequencies and damping ratios respectively for comparisons of the floors with the deck fixed to the joists either by screws or adhesives in addition to screws. For most comparisons, the frequencies of the different modes were slightly or moderately increasing when the bonding was used, apart from Mode (1,4) for comparison of Floors 2 I and M. The comparisons of Floors G and E of both test series were not following the general trend. These were the floors where two decking layers were used.

The damping ratios of Modes (1,1) were decreasing whereas only four comparisons were possible for this mode. For the other modes, the variation of damping was inconclusive. Generally, the use of adhesives can lower the amount of friction and thus the energy dissipation.

The comparisons in Figure 5.7 show a considerable reduction of deflection at the floor centre with a decrease between 25.00% and 37.50% if adhesive was used in addition to screws for fixing the deck to the joists. The only odd behaviour was again found from the comparison of Floors E and G of both test series. Their aberrant trend was, however, rather in agreement with the trend observed for the frequencies of these floors.

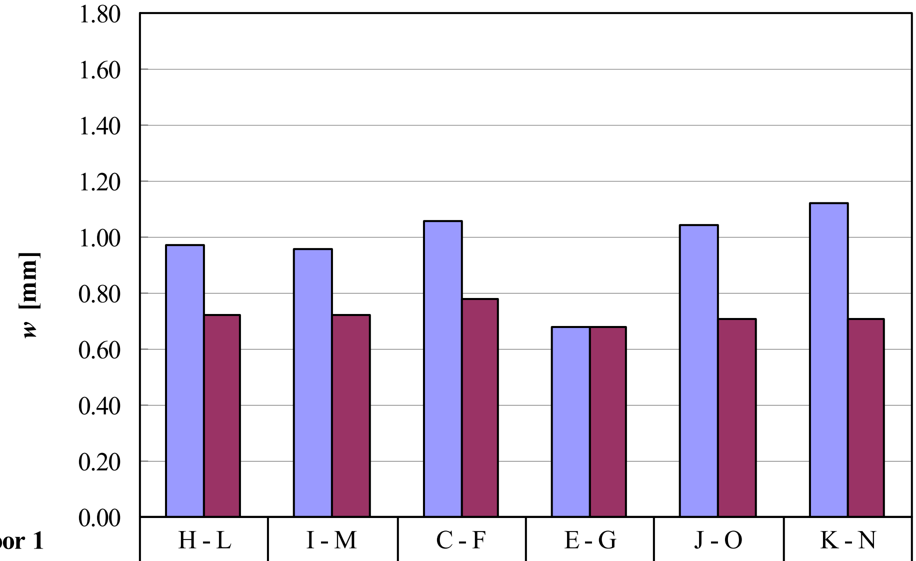
The bonding raised the degree of composite action of deck and joists, which enhances the shear behaviour and therefore the stiffness. This explains general increases in natural frequencies and reduction in the point load deflections.

Table 5.9: Natural frequencies of floors with varying deck to joist fixing method

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 H	17.02	23.00	27.76	31.09	34.80
1 L	18.08	24.06	28.63	32.23	35.93
Variation[%]	**+ 6.23**	**+ 4.61**	**+ 3.13**	**+ 3.67**	**+ 3.25**
1 I	17.55	23.56	28.27	31.24	34.93
1 M	18.08	24.07	29.09	32.24	36.11
Variation[%]	**+ 3.02**	**+ 2.16**	**+ 2.90**	**+ 3.20**	**+ 3.38**
1 C	-	25.89	32.32	34.45	37.51
1 F	-	26.68	33.35	36.37	41.48
Variation[%]		**+ 3.05**	**+ 3.19**	**+ 5.57**	**+ 10.58**
1 E	-	23.70	29.99	36.61	-
1 G	-	23.43	29.16	35.71	41.92
Variation[%]		**- 1.14**	**- 2.77**	**- 2.46**	
1 J	-	23.98	30.87	34.79	40.43
1 O	-	24.70	31.68	35.73	41.56
Variation[%]		**+ 3.00**	**+ 2.62**	**+ 2.70**	**+ 2.79**
1 K	-	23.20	29.32	34.63	40.07
1 N	-	23.75	30.72	35.53	40.79
Variation[%]		**+ 2.37**	**+ 4.77**	**+ 2.60**	**+ 1.80**
2 H	15.02	17.85	20.92	23.22	25.20
2 L	15.57	18.65	21.65	23.53	26.14
Variation[%]	**+ 3.66**	**+ 4.48**	**+ 3.49**	**+ 1.34**	**+ 3.73**
2 I	15.25	18.52	21.51	23.68	25.51
2 M	15.64	18.56	21.65	23.53	25.86
Variation[%]	**+ 2.56**	**+ 0.22**	**+ 0.65**	**- 0.63**	**+ 1.37**
2 C	-	20.42	22.89	-	-
2 F	-	20.79	24.08	26.25	27.15
Variation[%]		**+ 1.81**	**+ 5.20**		
2 E	-	19.12	24.22	28.26	-
2 G	-	18.52	23.49	27.99	-
Variation[%]		**- 3.14**	**- 3.01**	**- 0.96**	
2 K	-	19.38	23.00	26.48	29.64
2 N	-	19.74	23.53	27.42	30.03
Variation[%]		**+ 1.86**	**+ 2.30**	**+ 3.55**	**+ 1.32**
2 J	-	19.46	22.93	26.49	30.13
2 O	-	19.97	23.35	27.19	30.57
Variation[%]		**+ 2.62**	**+ 1.83**	**+ 2.64**	**+ 1.46**

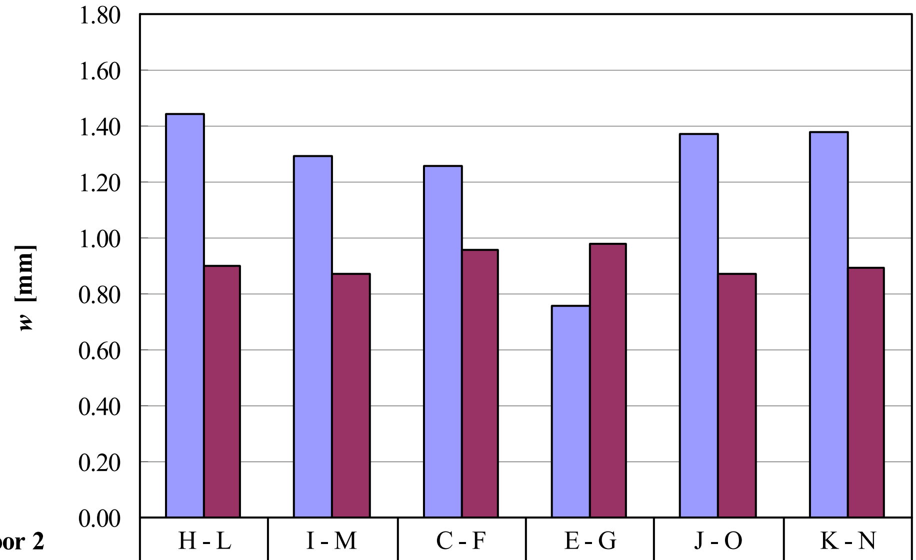
Table 5.10: Damping ratios of floors with varying deck to joist fixing method

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 H	2.60	1.19	1.07	0.97	1.13
1 L	2.04	1.32	0.98	1.20	1.26
Variation[%]	**- 21.54**	**+ 10.92**	**- 8.41**	**+ 23.71**	**+ 11.50**
1 I	2.45	1.21	1.46	1.33	1.51
1 M	2.33	1.22	1.07	1.12	1.08
Variation[%]	**- 4.90**	**+ 0.83**	**- 26.71**	**- 15.79**	**- 28.48**
1 C	-	2.15	1.98	1.42	1.74
1 F	-	2.55	1.95	1.30	1.68
Variation[%]		**+ 18.60**	**- 1.52**	**- 8.45**	**- 3.45**
1 E	-	1.82	1.64	1.45	-
1 G	-	1.86	1.36	1.64	2.14
Variation[%]		**+ 2.20**	**- 17.07**	**+ 13.10**	
1 J	-	2.02	1.29	1.11	1.94
1 O	-	2.07	1.49	1.45	1.34
Variation[%]		**+ 2.48**	**+ 15.50**	**+ 30.63**	**- 30.93**
1 K	-	2.18	1.15	1.45	1.92
1 N	-	2.09	1.08	1.32	1.66
Variation[%]		**- 4.13**	**- 6.09**	**- 8.97**	**- 13.54**
2 H	2.38	1.16	1.04	1.06	2.38
2 L	2.02	1.15	1.03	1.34	1.31
Variation[%]	**- 15.13**	**- 0.86**	**- 0.96**	**+ 26.42**	**- 44.96**
2 I	2.38	1.07	1.16	1.03	1.60
2 M	2.01	1.07	0.93	1.22	2.00
Variation[%]	**- 15.55**	**0.00**	**- 19.83**	**+ 18.45**	**+ 25.00**
2 C	-	1.82	1.78	-	-
2 F	-	1.85	1.54	1.08	2.16
Variation[%]		**+ 1.65**	**- 13.48**		
2 E	-	1.27	1.25	2.73	-
2 G	-	1.53	1.03	1.46	-
Variation[%]		**+ 20.47**	**- 17.60**	**- 46.52**	
2 K	-	1.74	1.48	1.36	1.70
2 N	-	1.58	1.64	1.73	1.41
Variation[%]		**- 9.20**	**+ 10.81**	**+ 27.21**	**- 17.06**
2 J	-	1.56	1.99	1.74	2.25
2 O	-	1.60	2.45	1.38	1.66
Variation[%]		**+ 2.56**	**+ 23.12**	**- 20.69**	**- 26.22**



Floor 1	H - L	I - M	C - F	E - G	J - O	K - N
Fasteners: Screws	0.97	0.96	1.06	0.68	1.04	1.12
Fasteners: Glue + screws	0.72	0.72	0.78	0.68	0.71	0.71

(a) Test series 1



Floor 2	H - L	I - M	C - F	E - G	J - O	K - N
Fasteners: Screws	1.44	1.29	1.26	0.76	1.37	1.38
Fasteners: Glue + screws	0.90	0.87	0.96	0.98	0.87	0.89

(b) Test series 2

Figure 5.7: Deflections at the floor centre for varying deck-to-joist fixing methods

5.1.4 Effect of floor span

The floor span was increased from 3.7 to 5.0 m while the joist depth was considerably raised by 51%, and thus the joist flexural rigidity by 175%, for the longer floors. The influence of increasing the floor span with increasing joist stiffness on natural frequencies and damping can be found from the test results listed in Table 5.11 and

Table 5.12. Without exception, all natural frequencies decreased by double percentage figures. Mode (1,1) was less affected with an average reduction of 15.33% (six comparisons) than the higher modes with average reductions between 20.70% for Mode (1,2) and 27.68% for Mode (1,5) (fifteen comparisons for both).

Table 5.11: Natural frequencies of floors with varying span

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	19.84	25.94	29.86	32.38	34.65
2 A	15.80	19.76	21.79	23.01	-
Variation[%]	**- 20.36**	**- 23.82**	**- 27.03**	**- 28.94**	
1 B	20.15	27.13	30.50	32.39	34.83
2 B	16.25	20.53	22.32	-	-
Variation[%]	**- 19.35**	**- 24.33**	**- 26.82**		
1 H	17.02	23.00	27.76	31.09	34.80
2 H	15.02	17.85	20.92	23.22	25.20
Variation[%]	**- 11.75**	**- 22.39**	**- 24.64**	**- 25.31**	**- 27.59**
1 I	17.55	23.56	28.27	31.24	34.93
2 I	15.25	18.52	21.51	23.68	25.51
Variation[%]	**- 13.11**	**- 21.39**	**- 23.91**	**- 24.20**	**- 26.97**
1 L	18.08	24.06	28.63	32.23	35.93
2 L	15.57	18.65	21.65	23.53	26.14
Variation[%]	**- 13.88**	**- 22.49**	**- 24.38**	**- 26.99**	**- 27.25**
1 M	18.08	24.07	29.09	32.24	36.11
2 M	15.64	18.56	21.65	23.53	25.86
Variation[%]	**- 13.50**	**- 22.89**	**- 25.58**	**- 27.02**	**- 28.39**
1 C	-	25.89	32.32	34.45	37.51
2 C	-	20.42	22.89	-	-
Variation[%]		**- 21.13**	**- 29.18**		
1 D	-	24.46	31.80	34.11	37.46
2 D	-	19.97	22.89	-	-
Variation[%]		**- 18.36**	**- 28.02**		
1 E	-	23.70	29.99	36.61	-
2 E	-	19.12	24.22	28.26	-
Variation[%]		**- 19.32**	**- 19.24**	**- 22.81**	
1 F	-	26.68	33.35	36.37	41.48
2 F	-	20.79	24.08	26.25	27.15
Variation[%]		**- 22.08**	**- 27.80**	**- 27.83**	**- 34.55**
1 G	-	23.43	29.16	35.71	41.92
2 G	-	18.52	23.49	27.99	-
Variation[%]		**- 20.96**	**- 19.44**	**- 21.62**	
1 J	-	23.98	30.87	34.79	40.43
2 J	-	19.46	22.93	26.49	30.13
Variation[%]		**- 18.85**	**- 25.72**	**- 23.86**	**- 25.48**
1 K	-	23.20	29.32	34.63	40.07
2 K	-	19.38	23.00	26.48	29.64
Variation[%]		**- 16.47**	**- 21.56**	**- 23.53**	**- 26.03**
1 N	-	23.75	30.72	35.53	40.79
2 N	-	19.74	23.53	27.42	30.03
Variation[%]		**- 16.88**	**- 23.40**	**- 22.83**	**- 26.38**
1 O	-	24.70	31.68	35.73	41.56
2 O	-	19.97	23.35	27.19	30.57
Variation[%]		**- 19.15**	**- 26.29**	**- 23.90**	**- 26.44**

Table 5.12: Damping ratios of floors with varying span

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 A	3.38	1.43	1.26	1.45	1.47
2 A	3.13	1.39	1.76	1.93	-
Variation[%]	**- 7.40**	**- 2.80**	**+ 39.68**	**+ 33.10**	
1 B	2.55	1.19	1.16	1.41	1.59
2 B	2.32	1.16	1.63	-	-
Variation[%]	**- 9.02**	**- 2.52**	**+ 40.52**		
1 H	2.60	1.19	1.07	0.97	1.13
2 H	2.38	1.16	1.04	1.06	2.38
Variation[%]	**- 8.46**	**- 2.52**	**- 2.80**	**+ 9.28**	**+ 110.62**
1 I	2.45	1.21	1.46	1.33	1.51
2 I	2.38	1.07	1.16	1.03	1.60
Variation[%]	**- 2.86**	**- 11.57**	**- 20.55**	**- 22.56**	**+ 5.96**
1 L	2.04	1.32	0.98	1.20	1.26
2 L	2.02	1.15	1.03	1.34	1.31
Variation[%]	**- 0.98**	**- 12.88**	**+ 5.10**	**+ 11.67**	**+ 3.97**
1 M	2.33	1.22	1.07	1.12	1.08
2 M	2.01	1.07	0.93	1.22	2.00
Variation[%]	**- 13.73**	**- 12.30**	**- 13.08**	**+ 8.93**	**+ 85.19**
1 C	-	2.15	1.98	1.42	1.74
2 C	-	1.82	1.78	-	-
Variation[%]		**- 15.35**	**- 10.10**		
1 D	-	2.40	1.99	1.77	1.66
2 D	-	1.99	1.81	-	-
Variation[%]		**- 17.08**	**- 9.05**		
1 E	-	1.82	1.64	1.45	-
2 E	-	1.27	1.25	2.73	-
Variation[%]		**- 30.22**	**- 23.78**	**+ 88.28**	
1 F	-	2.55	1.95	1.30	1.68
2 F	-	1.85	1.54	1.08	2.16
Variation[%]		**- 27.45**	**- 21.03**	**- 16.92**	**+ 28.57**
1 G	-	1.86	1.36	1.64	2.14
2 G	-	1.53	1.03	1.46	-
Variation[%]		**- 17.74**	**- 24.26**	**- 10.98**	
1 J	-	2.02	1.29	1.11	1.94
2 J	-	1.56	1.99	1.74	2.25
Variation[%]		**- 22.77**	**+ 54.26**	**+ 56.76**	**+ 15.98**
1 K	-	2.18	1.15	1.45	1.92
2 K	-	1.74	1.48	1.36	1.70
Variation[%]		**- 20.18**	**+ 28.70**	**- 6.21**	**- 11.46**
1 N	-	2.09	1.08	1.32	1.66
2 N	-	1.58	1.64	1.73	1.41
Variation[%]		**- 24.40**	**+ 51.85**	**+ 31.06**	**- 15.06**
1 O	-	2.07	1.49	1.45	1.34
2 O	-	1.60	2.45	1.38	1.66
Variation[%]		**- 22.71**	**+ 64.43**	**- 4.83**	**+ 23.88**

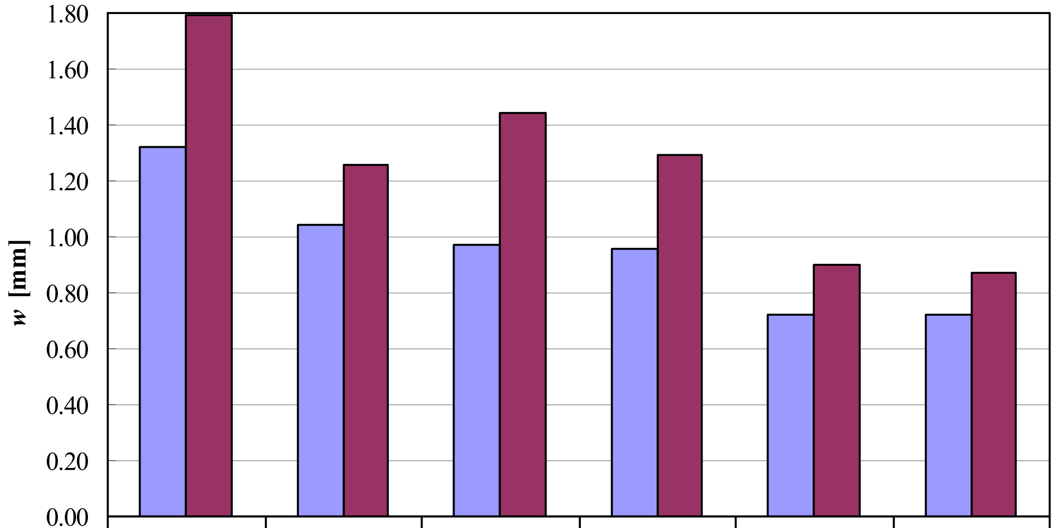
For the higher modes the degree of the effect often was about the same or slightly increased for the next higher mode. The absolute spacing of the natural frequencies became closer. The spacing of the frequencies between Mode (1,1) and Mode (1,2) fell below the critical value of 5 HZ, which was defined by Ohlsson (1982). This happened mainly also for the spacing of the frequencies of higher modes if it was not already critical.

The damping decreased for all comparisons of Modes (1,1) and (1,2). For higher modes, the variation was rather inconclusive. The decrease in damping corresponding to the first vibration mode lay at an average of 7.08%. Comparing the damping ratios of Mode (1,2) shows higher influence on the floors supported along four sides instead of two sides. The floors decked with a single OSB layer were usually less affected than those decked with a single particleboard layer or double layer of OSB and particleboard.

The deflection was ascending by double percentage figures of mainly more than 20% comparing the longer to the shorter floors (Figure 5.8). One result indicated no variation in deflection.

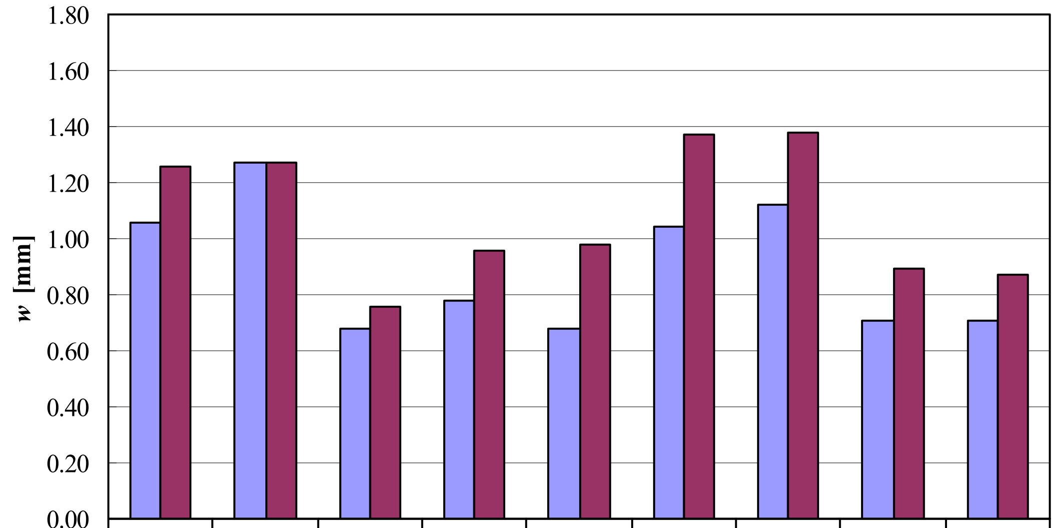
5.1.5 Discussion on the results of TJI floor test series

The influence of variations with regard to floor deck, floor span, boundary conditions and deck to joist fixing method had been investigated. Although deflection tests were repeated and inspection measures installed, uncertainties in the results occurred. This could have been caused by the provision of access to the floor for loading and unloading, the use of different floor samples and potential misplacement of the load, which could result in unintended local deflection when the load was not fully in line with the joists. Some of the odd-appearing deflection results were however in line with the results of the vibration tests (varying decks and deck-to-joist connection methods), which could give an indication of stiffness variation. As was explained earlier, different floor samples had to be used, which may have contributed to some uncertainties.



Floor	1A - 2A	1B - 2B	1H - 2H	1I - 2I	1L - 2L	1M - 2M
L = 3.7 m	1.32	1.04	0.97	0.96	0.72	0.72
L = 5.0 m	1.79	1.26	1.44	1.29	0.90	0.87

(a) Floors supported at two sides



Floor	1C - 2C	1D - 2D	1E - 2E	1F - 2F	1G - 2G	1J - 2J	1K - 2K	1N - 2N	1O - 2O
L = 3.7 m	1.06	1.27	0.68	0.78	0.68	1.04	1.12	0.71	0.71
L = 5.0 m	1.26	1.27	0.76	0.96	0.98	1.37	1.38	0.89	0.87

(b) Floors supported at four sides

Figure 5.8: Deflections at the floor centre of floors with varying spans

Significant variation in the mode shapes were found for the comparison of two- and four-side supported floors, as could be expected. The individual mode shapes showed all considerable movement at unsupported edges. Providing supports under these edges reduced the movement to a rather insignificant level, possibly to be even close to behaving like a node. The remaining movements at these locations contributed to the friction between flooring structure and supports, leading to generally enlarged energy dissipation. This was confirmed by the damping ratios of Modes (1,2) to (1,5) as they mainly considerably increased due to the additional supports. A raise in damping could therefore also be expected for Mode (1,1), which was not definitely identifiable for the floors supported along all four edges. A possible relative high damping could explain lowering of the amplitude of Mode (1,1). An elimination of the appropriate peak in the frequency spectrum was not expected. The testing method and analysis were excluded as the cause, since different methods were used. It is possible that Mode (1,1) was covered by Mode (1,2), which is considered to become fundamental, and thus not found. However, the potential increase in the frequency would have been enormous, larger than expected. While an increase in the first natural frequency is usually considered beneficial, two closely spaced modes can undesirably interact so as to produce relatively high amplitudes. Otherwise the natural frequencies increased with higher efficiency on higher modes, leading to a wider separation of the more closely spaced higher modes. The efficiency of added supports on the ascent in natural frequencies was mainly raised when stiffer flooring materials were used. Although the influence of Mode (1,1) on the vibration performance could not be fully determined, it is recommendable to support flooring systems along all sides whenever possible.

If a floor is found to exhibit unacceptable vibration behaviour and Mode (1,1) is not identifiable, Mode (1,2) becomes fundamental. For two- and four-side supported floors, this mode has a node along the floor centre in longitudinal direction. When conducting a vibration test, the response at this location may not be dominating but could be misleadingly taken as the response of the fundamental mode. This could happen when only a measurement point at the centre of the floor is selected, and mode shapes not identified. This is sometimes practice for investigating e.g. acceleration responses and the natural frequency at the assumed anti-node of the fundamental mode. Misinterpretation of results could be avoided by taking measurements at least at assumed anti-nodes of Mode (1,2) additionally, or by using a larger number of measurement points allowing the identification of the mode shapes (in transverse direction).

The generally ignorable variation of deflection and the inconclusiveness of some results for adding supports to the floors confirmed the assumption that the deflection measured at the centre of relatively wide floors could be valid for the two cases of floors being supported along two or all four edges even though it is determined only for one of these support types.

The effect of raising the support end fixity by screw-fixing the floor to the supports as applied for the investigations in these test series was insignificant. The adopted method was therefore thought to be close to the condition of simple supports. Thus, it would be rather inefficient to use this method in practice since the end fixity of floors could nevertheless slightly rise if they were finally installed between walls in buildings.

The investigations of three different decking types showed different influences for lower and higher frequencies. Using the 22 mm thick particleboard deck as a replacement of the 15 mm thick OSB deck or on top of it increased the stiffness in both directions and simultaneously raised the mass. Regarding the natural frequencies, the results indicated that the mass effect was dominating at the lower modes with reduced frequencies, and the stiffness effect became more influential on successive modes by raising frequencies. The mass was less influential on the natural frequencies of the larger floors. This could have been caused by the higher flexural rigidity of the joists used for the larger floors. The damping in general decreased especially for Modes (1,1) to (1,3) and in particular for the floors decked with two layers.

It is therefore recommendable that a deck should be used for the flooring structure that meets the static and acoustic requirements and which is found to be acceptable regarding the degree of local deflections at areas between two adjacent joists. It should not be oversized as the mass effect could dominate at the lower natural frequencies and also the damping could be reduced. Indeed, the double deck layer showed best performance in the deflection tests as the stiffness is the controlling factor there. However, adhesives used in addition to screws for fixing the single deck to the joists yielded results approaching those of the floors with two decking layers. Also the use of adhesives could lower damping but otherwise would raise the degree of composite action and thus stiffness and has no noteworthy effect on mass.

To be more precise, the use of adhesives can be considered to moderately increase the natural frequencies due to the raised stiffness and to lower the damping, particularly the one corresponding to Mode (1,1). The increase in stiffness is achieved by the increase in the degree of composite action due to the bonding, which enhances the shear behaviour of deck and joists. However, the lowered damping could have been caused by this effect as energy dissipation due to friction was thereby reduced. The deflection of floors with single layer deck could be notably reduced, in the tests by 25.00% to 37.50%.

Increasing the span from 3.7 to 5.0 m while strongly raising the flexural rigidity of the joists for the larger floors reduced the natural frequencies significantly. The increase in stiffness could not balance the effect due to the raised span. A closer separation of the natural frequencies, which was below the critical value for adjacent modes (assumed to be 5 Hz according to Ohlsson (1982)), was usually found. The frequency spacing especially of Modes (1,1) and (1,2) was not critical for the shorter floors. When the ratio of L/B is increasing, a growth in the spacing of neighbouring frequencies can usually be expected. The contrary behaviour observed can be explained by the simultaneous increase in joist stiffness. When the joist stiffness is increased, the ratio of $(EI)_x/(EI)_y$ is decreasing, which usually results in a reduced spacing of adjacent natural frequencies. The effect of the variation of the stiffness ratio outperformed the variation of the length to width ratio, finally resulting in a lowered separation of adjacent natural frequencies. As longer floors usually require stiffer joists than shorter floors, a decrease in frequency spacing can therefore be expected. The damping of the Modes (1,1) and (1,2) was lowered by on average 7.08% and 16.17% respectively. The deflection was usually raised by double percentage figures.

All damping ratios corresponding to Mode (1,1) of all flooring structures were found to be above 2%. The damping ratios of all other modes were mainly below 2%. It is noteworthy that also most damping ratios of Mode (1,2) of the floors of Test series 1, which were supported at four sides, had damping ratios above 2%, apart from two, which were the floors with double layer deck. The damping ratios of the same mode for four-side supported floors of Test series 2 were all below 2%, with the lowest values found again for the floors with double layer deck. This trend was generally noted in the tests with different decking configurations whereas the raised amount of decking material generally reduced the damping of the Modes (1,1) to (1,3). Damping characteristics are examined further in Chapter 8.

5.2 Results of the JJI floor test series

The vibration test results of Test series 1 and 2 are presented in Table 5.13 and Table 5.14 respectively. Furthermore, the frequencies and damping of the floors are compared in tabular format in the Sections 5.2.1 to 5.2.7. It was focused on the first five principal modes of vibration. Modes which could not be fully identified are not included. The results of the deflection tests are presented in Table 5.16 and Table 5.15 and also compared in tabular format in the following subsections. One of the flooring systems was additionally selected for investigations in connection with the STSM of COST Action E55 at VTT. The results of this are presented in Chapter 7.

5.2.1 Effect of varied boundary conditions

In this study, the method to connect the floor to the supports was different from the one investigated on the TJI flooring structures. However, the trend was similar with slightly increasing frequencies (Table 5.17), decreasing damping for the lower modes (Table 5.18) and little effect on the deflection (Table 5.19).

Table 5.13: Vibration test results of Test series JJI 1

Floor 1	$f_{(1,1)}$ [Hz]	$\zeta_{(1,1)}$ [%]	$f_{(1,2)}$ [Hz]	$\zeta_{(1,2)}$ [%]	$f_{(1,3)}$ [Hz]	$\zeta_{(1,3)}$ [%]	$f_{(1,4)}$ [Hz]	$\zeta_{(1,4)}$ [%]	$f_{(1,5)}$ [Hz]	$\zeta_{(1,5)}$ [%]
A	24.46	3.98	30.11	0.95	35.58	1.08	47.33	1.13	60.72	1.16
B	16.26	1.57	17.69	1.05	19.25	2.39	20.53	2.22	-	-
C	13.48	1.09	14.57	1.52	16.69	3.25	18.67	3.28	-	-
D	25.23	2.51	30.99	1.64	36.45	0.96	48.53	0.98	59.34	1.04
E	16.62	1.37	17.62	1.59	19.60	1.53	22.74	2.22	-	-
F	13.62	1.26	14.25	1.12	16.29	4.18	18.53	2.31	-	-
G	27.29	3.98	30.84	1.03	39.17	1.04	51.38	1.13	60.63	1.10
H	17.37	1.25	17.89	1.38	21.86	1.37	22.68	3.05	-	-
I	14.08	1.43	14.73	1.37	18.74	3.76	19.59	3.02	-	-
J	25.43	3.56	30.26	1.00	36.86	0.95	51.55	1.64	60.39	1.02
K	16.76	1.48	17.49	1.14	19.78	1.00	-	-	-	-
L	13.85	1.11	14.30	1.18	16.78	3.56	18.77	2.84	-	-
M	24.66	3.32	32.26	1.70	37.28	1.96	50.36	1.07	61.16	0.97
N	26.97	2.80	33.25	1.04	37.93	1.28	55.06	1.16	62.49	0.94
O	27.37	3.48	34.49	1.25	39.89	1.98	51.79	0.87	62.81	0.94
P	28.10	3.16	35.54	1.10	-	-	52.41	1.03	62.18	0.91

Table 5.14: Vibration test results of Test series JJI 2

Floor 2	$f_{(1,1)}$ [Hz]	$\zeta_{(1,1)}$ [%]	$f_{(1,2)}$ [Hz]	$\zeta_{(1,2)}$ [%]	$f_{(1,3)}$ [Hz]	$\zeta_{(1,3)}$ [%]	$f_{(1,4)}$ [Hz]	$\zeta_{(1,4)}$ [%]	$f_{(1,5)}$ [Hz]	$\zeta_{(1,5)}$ [%]
A	22.66	2.41	28.81	0.99	34.88	0.80	47.65	1.10	59.63	1.02
A**	22.49	2.64	28.58	1.04	33.98	1.12	45.87	0.91	58.27	0.96
G	24.89	3.20	28.57	1.02	40.17	1.13	45.51	0.90	57.81	1.34
J	23.87	2.71	28.58	0.97	35.34	0.99	49.79	1.19	58.85	1.03
M	22.77	2.93	31.49	0.96	36.97	0.96	48.99	1.18	61.33	1.08
N	23.98	2.74	31.57	0.85	38.22	1.74	51.84	1.15	61.99	1.06
O	25.09	3.00	32.39	0.93	39.67	0.99	51.63	1.01	60.86	1.18
P	27.02	3.85	33.96	0.88	40.87	1.68	52.61	0.96	61.85	1.63
Q	22.11	2.79	27.55	1.04	33.82	0.94	47.04	1.06	57.52	0.86
R	14.68	1.31	15.99	0.98	18.39	1.34	19.85	1.25	-	-
S	12.16	1.25	13.19	1.06	15.32	1.24	19.32	3.08	-	-
T	22.59	2.51	28.98	0.97	45.03	1.31	57.17	1.02	-	-
U	22.23	2.21	28.55	0.97	48.24	1.31	58.86	1.05	-	-

Table 5.15: Deflections of floors JJI 1

Floor 1	w_A [mm]	w_B [mm]	w_C [mm]	w_D [mm]
A	1.31	1.38	1.75	-
B	1.29	-	-	-
C	1.29	-	-	-
D	0.90	1.35	-	-
E	0.92	-	-	-
F	0.93	-	-	-
G	0.42	1.24	-	-
H	0.43	-	-	-
I	0.43	-	-	-
J	0.72	1.32	-	-
K	0.73	-	-	-
L	0.71	-	-	-
M	1.37	1.34	-	-
N	0.71	1.22	-	-
O	1.01	0.99	1.09	-
P	0.91	-	0.93	0.91

Table 5.16: Deflections of floors JJI 2

Floor 2	w_A [mm]	w_B [mm]	w_C [mm]	w_D [mm]
A	1.59	1.68	-	-
G	0.56	1.47	-	-
J	0.85	1.35	-	-
M	1.57	1.38	-	-
N	0.80	1.33	-	-
O	1.14	-	-	-
P	0.91	-	-	-
Q	1.58	-	-	-
R	-	-	-	-
S	1.51	-	-	-
T	1.20	1.22	-	-
U	1.16	1.03	-	-

Table 5.17: Natural frequencies due to increased support end fixity

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
2 Q	22.11	27.55	33.82	47.04	57.52
2 A	22.66	28.81	34.88	47.65	59.63
Variation [%]	**+ 2.49**	**+ 4.57**	**+ 3.13**	**+ 1.30**	**+ 3.67**

Table 5.18: Damping ratios due to increased support end fixity

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
2 Q	2.79	1.04	0.94	1.06	0.86
2 A	2.41	0.99	0.80	1.10	1.02
Variation [%]	**- 13.62**	**- 4.81**	**- 14.89**	**+ 3.77**	**+ 18.60**

Table 5.19: Deflections at floor centre due to increased support end fixity

Floor	w_A [mm]
2 Q	1.58
2 A	1.59
Variation [%]	**+ 0.63**

5.2.2 Effect of varied joist spacing

Table 5.20 and Table 5.21 show the comparison of the natural frequencies and damping ratios respectively for Modes (1,1) to (1,5) for reducing the joist spacing from 600 mm to 400 mm or to 300 mm. The natural frequencies of the principal modes identified were increasing as a result of raised stiffness when the spacing between joists was lowered. The fundamental natural frequency was raised by 14.88% (FJ 1 P) and 19.24% (FJ 2 P) when halving the joist spacing. The frequency of Mode (1,5) was raised by up to 3.72%. The spacing between frequencies sometimes increased for lower modes but usually decreased for higher modes. A reduction in the separation of adjacent natural frequencies can be expected from decreasing the joist spacing as this raises the degree of orthotropy of the floor.

The damping ratios yielded inconclusive results. When reducing the joist spacing, the damping ratios corresponding to Modes (1,1) and (1,5) decreased in Test series 1 but increased in Test series 2. In case of Mode (1,2), the trend was reverse for the individual test series. Damping of Mode (1,3) was raised and of Mode (1,4) lowered.

Table 5.22 shows the variation in deflection. When the joist spacing was reduced from 600 mm to 400 mm or to 300 mm, the deflection at the floor centre was clearly lowered by 22.90% and 30.53% for Test series 1 and by 28.30% and 42.77% for Test series 2 respectively.

Table 5.20: Natural frequencies due to reduced joist spacing

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	24.46	30.11	35.58	47.33	60.72
1 O	27.37	34.49	39.89	51.79	62.81
Variation [%]	**+ 11.90**	**+ 14.55**	**+ 12.11**	**+ 9.42**	**+ 3.44**
1 P	28.10	35.54	-	52.41	62.18
Variation [%]	**+ 14.88**	**+ 18.03**		**+ 10.73**	**+ 2.40**
2 A	22.66	28.81	34.88	47.65	59.63
2 O	25.09	32.39	39.67	51.63	60.86
Variation [%]	**+ 10.72**	**+ 12.43**	**+ 13.73**	**+ 8.35**	**+ 2.06**
2 P	27.02	33.96	40.87	52.61	61.85
Variation [%]	**+ 19.24**	**+ 17.88**	**+ 17.17**	**+ 10.41**	**+ 3.72**

Table 5.21: Damping ratios due to reduced joist spacing

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 A	3.98	0.95	1.08	1.13	1.16
1 O	3.48	1.25	1.98	0.87	0.94
Variation [%]	**- 12.56**	**+ 31.58**	**+ 83.33**	**- 23.01**	**- 18.97**
1 P	3.16	1.10	-	1.03	0.91
Variation [%]	**- 20.60**	**+ 15.79**		**- 8.85**	**- 21.55**
2 A	2.41	0.99	0.80	1.10	1.02
2 O	3.00	0.93	0.99	1.01	1.18
Variation [%]	**+ 24.48**	**- 6.06**	**+ 23.75**	**- 8.18**	**+ 15.69**
2 P	3.85	0.88	1.68	0.96	1.63
Variation [%]	**+ 59.75**	**- 11.11**	**+ 110.00**	**- 12.73**	**+ 59.80**

Table 5.22: Deflections at floor centre due to reduced joist spacing

Floor	w_A [mm]
1 A	1.31
1 O	1.01
Variation [%]	**- 22.90**
1 P	0.91
Variation [%]	**- 30.53**
2 A	1.59
2 O	1.14
Variation [%]	**- 28.30**
2 P	0.91

5.2.3 Effect of I-joist blocking

The effect of installing I-joist blocking on natural frequencies and damping ratios is presented in Table 5.23 and Table 5.24 respectively. The use of I-joist blocking had no noteworthy effect on the frequencies of Modes (1,1) and (1,2). The frequencies of Mode (1,3) were raised by 29.10% for Floor 2 T and 38.30% for Floor 2 U compared to Floor 2 A, those of Mode (1,4) by 19.98% and 23.53% respectively. Therefore, the frequency spacing between Modes (1,2) and (1,3) was clearly raised whereas the one between Modes (1,3) and (1,4) was slightly reduced. There was a higher efficiency found for higher number of rows of I-joist blocking.

The variation in damping was rather inconclusive. However, the damping ratios were about the same for the same mode numbers, apart from the damping corresponding to Mode (1,3) of Floor 2 A.

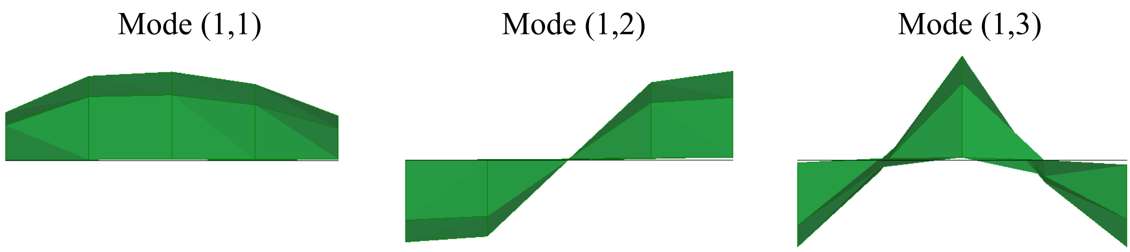
Table 5.23: Natural frequencies due to the use of I-joist blocking

Floor	$f_{(1,1)}$ **[Hz]**	$f_{(1,2)}$ **[Hz]**	$f_{(1,3)}$ **[Hz]**	$f_{(1,4)}$ **[Hz]**	$f_{(1,5)}$ **[Hz]**
2 A	22.66	28.81	34.88	47.65	59.63
2 T	22.59	28.98	45.03	57.17	-
Variation [%]	**- 0.31**	**+ 0.59**	**+ 29.10**	**+ 19.98**	
2 U	22.23	28.55	48.24	58.86	-
Variation [%]	**- 1.90**	**- 0.90**	**+ 38.30**	**+ 23.53**	

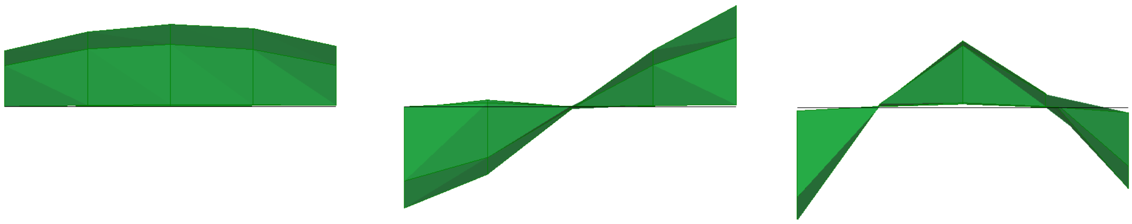
Table 5.24: Damping ratios due to the use of I-joist blocking

Floor	$\zeta_{(1,1)}$ **[%]**	$\zeta_{(1,2)}$ **[%]**	$\zeta_{(1,3)}$ **[%]**	$\zeta_{(1,4)}$ **[%]**	$\zeta_{(1,5)}$ **[%]**
2 A	2.41	0.99	0.80	1.10	1.02
2 T	2.51	0.97	1.31	1.02	-
Variation [%]	**+ 4.15**	**- 2.02**	**+ 63.75**	**- 7.27**	
2 U	2.21	0.97	1.31	1.05	-
Variation [%]	**- 8.30**	**- 2.02**	**+ 63.75**	**- 4.55**	

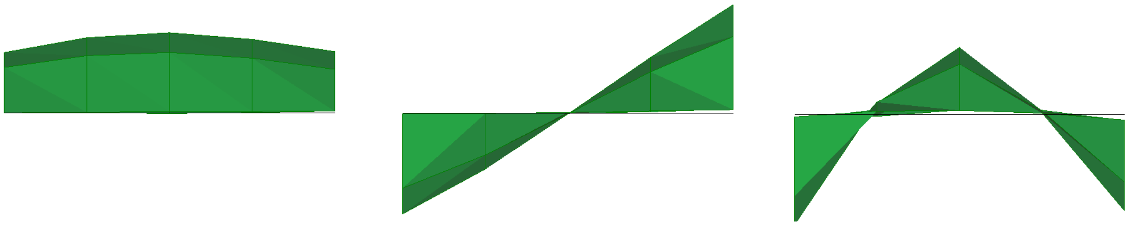
The mode shapes in transverse direction became flatter for Mode (1,1) and also less curved for Mode (1,2) for inserting I-joist blocking at mid-span. The effect was amplified for I-joist blocking at third-spans. The movement at the edges became more significant. The movement in the centre of Mode (1,3) was maximum for the floor without blocking and was lower than at the edges when blocking was used (Figure 5.9).



(a) Floor 2 A (without I-joist blocking)



(b) Floor 2 T (with one row of I-joist blocking at mid-span)



(c) Floor 2 U (with one row of I-joist blocking at each third-span)

Figure 5.9: Shapes of Modes (1,1) to (1,3) for floors with and without I-joist blocking

The deflection at the point of application was lowered by about a quarter at the floor centre and to a higher degree at mid-span of the adjacent joist since the use of blocking increased the transfer of the load to other joists. Using blocking at the third-spans reduced the deflection more effectively (Table 5.25) whereas, oddly, the joist under loading adjacent to the unsupported edge deflected less than the joist under loading at the floor centre.

Table 5.25: Deflections due to the use of I-joist blocking

Floor	**w_A [mm]**	**w_B [mm]**
2 A	1.59	1.68
2 T	1.20	1.22
Variation [%]	**- 24.53**	**- 27.38**
2 U	1.16	1.03
Variation [%]	**- 27.04**	**- 38.69**

5.2.4 Effect of joist depth

Increasing the joist depth by varying the web height mainly raised frequencies, for Mode (1,1) by an average of 8.28% and for Mode (1,2) by an average of 5.32% (Table 5.26). The variation of Mode (1,3) was little, with an average increase of 0.74%. The variation of Mode (1,4) was no more than 0.67% for the floors with regular joist arrangements. For the floors with double joists, however, the frequencies of this mode increased, notably by 6.21% for Floors N and 12.90% for Floors G. Those two floors also exhibited the highest rise in the fundamental frequency. The spacing between adjacent natural frequencies was narrowed in general for Modes (1,1) to (1,4) but increased between Modes (1,3) and (1,4) for all floors with double joists. In turn, the spacing between the natural frequencies of Modes (1,4) and (1,5) of the floors with double joists was lowered while some increase in frequency separation of those modes was found for the other floors (see Section 5.2.8 for discussion).

In six out of seven comparisons the damping ratios increased to varying degrees for Modes (1,1) and (1,2) and decreased for Mode (1,5) (Table 5.27). The variation was inconclusive otherwise.

Table 5.26: Natural frequencies due to increased joist depth

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
2 A	22.66	28.81	34.88	47.65	59.63
1 A	24.46	30.11	35.58	47.33	60.72
Variation [%]	**+ 7.94**	**+ 4.51**	**+ 2.01**	**- 0.67**	**+ 1.83**
2 G	24.89	28.57	40.17	45.51	57.81
1 G	27.29	30.84	39.17	51.38	60.63
Variation [%]	**+ 9.64**	**+ 7.95**	**- 2.49**	**+ 12.90**	**+ 4.88**
2 J	23.87	28.58	35.34	49.79	58.85
1 J	25.43	30.26	36.86	51.55	60.39
Variation [%]	**+ 6.54**	**+ 5.88**	**+ 4.30**	**+ 3.53**	**+ 2.62**
2 M	22.77	31.49	36.97	48.99	61.33
1 M	24.66	32.26	37.28	50.36	61.16
Variation [%]	**+ 8.30**	**+ 2.45**	**+ 0.84**	**+ 2.80**	**- 0.28**
2 N	23.98	31.57	38.22	51.84	61.99
1 N	26.97	33.25	37.93	55.06	62.49
Variation [%]	**+ 12.47**	**+ 5.32**	**- 0.76**	**+ 6.21**	**+ 0.81**
2 O	25.09	32.39	39.67	51.63	60.86
1 O	27.37	34.49	39.89	51.79	62.81
Variation [%]	**+ 9.09**	**+ 6.48**	**+ 0.55**	**+ 0.31**	**+ 3.20**
2 P	27.02	33.96	40.87	52.61	61.85
1 P	28.10	35.54	-	52.41	62.18
Variation [%]	**+ 4.00**	**+ 4.65**		**- 0.38**	**+ 0.53**

Table 5.27: Damping ratios due to increased joist depth

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
2 A	2.41	0.99	0.80	1.10	1.02
1 A	3.98	0.95	1.08	1.13	1.16
Variation [%]	**+ 65.15**	**- 4.04**	**+ 35.00**	**+ 2.73**	**+ 13.73**
2 G	3.20	1.02	1.13	0.90	1.34
1 G	3.98	1.03	1.04	1.13	1.10
Variation [%]	**+ 24.38**	**+ 0.98**	**- 7.96**	**+ 25.56**	**- 17.91**
2 J	2.71	0.97	0.99	1.19	1.03
1 J	3.56	1.00	0.95	1.64	1.02
Variation [%]	**+ 31.37**	**+ 3.09**	**- 4.04**	**+ 37.82**	**- 0.97**
2 M	2.93	0.96	0.96	1.18	1.08
1 M	3.32	1.70	1.96	1.07	0.97
Variation [%]	**+ 13.31**	**+ 77.08**	**+ 104.17**	**- 9.32**	**- 10.19**
2 N	2.74	0.85	1.74	1.15	1.06
1 N	2.80	1.04	1.28	1.16	0.94
Variation [%]	**+ 2.19**	**+ 22.35**	**- 26.44**	**+ 0.87**	**- 11.32**
2 O	3.00	0.93	0.99	1.01	1.18
1 O	3.48	1.25	1.98	0.87	0.94
Variation [%]	**+ 16.00**	**+ 34.41**	**+ 100.00**	**- 13.86**	**- 20.34**
2 P	3.85	0.88	1.68	0.96	1.63
1 P	3.16	1.10	-	1.03	0.91
Variation [%]	**- 17.92**	**+ 25.00**		**+ 7.29**	**- 44.17**

The deflection at floor centre was decreasing by double percentage figures for using deeper floor joists with exception of the floors with 300 mm joist spacing where no variation was found in the tests. Measuring the deflection at mid-span of the adjacent joist under load showed reduced deflection of varying degrees for the tested structures (Table 5.28).

5.2.5 Effect of targeted stiffening sensitive locations

Results from Test series 2 for using stiffer single or double joists at sensitive locations were presented in Weckendorf et al (2008a). As the corresponding mode shapes were representative examples of the findings, they were also selected for illustration in this section. The natural frequencies are presented in Table 5.29, the corresponding damping ratios in Table 5.30, and the deflections in Table 5.31.

Table 5.28: Deflections due to increased joist depth

Floor	w_A [mm]	w_B [mm]
2 A	1.59	1.68
1 A	1.31	1.38
Variation [%]	**- 17.61**	**- 17.86**
2 G	0.57	1.47
1 G	0.42	1.24
Variation [%]	**- 26.32**	**- 15.65**
2 J	0.85	1.35
1 J	0.72	1.32
Variation [%]	**- 15.29**	**- 2.22**
2 M	1.57	1.38
1 M	1.37	1.34
Variation [%]	**- 12.74**	**- 2.90**
2 N	0.81	1.33
1 N	0.71	1.22
Variation [%]	**- 12.35**	**- 8.27**
2 O	1.14	-
1 O	1.01	0.99
Variation [%]	**- 11.40**	
2 P	0.91	-
1 P	0.91	-
Variation [%]	**0.00**	

To illustrate the effect of double joists more comprehensibly, the variation in modal shapes is presented before the change in natural frequencies is described. The measure of using double joists was undertaken with focus on the benefit of the first three principal modes, but some beneficial manipulation was finally also identified with respect to the higher ones. The mode shapes in transverse direction corresponding to the floors of Test series 2 are illustrated in Figure 5.10 and Figure 5.11 which includes the different variations with double joists. The mode shapes of Floor 1 N are included (Figure 5.10(f) and Figure 5.11(f)) as the shape of Mode (1,3) of Floor 2 N was inconclusive. As apart from that likewise effects were found regarding the mode shapes in Test series 1 for the same kind of structural modifications, they were not additionally included in the figure.

The test results show that the double joists used along the central line lowered the modal displacements at this location largely in relative terms with respect to the cross sections of the mode shapes for Mode (1,1) and notably for Modes (1,3) and (1,5) compared to those of the base floor. This effect can especially be seen for Floor 2 G where the lowest displacement was observed at the centre of the whole cross section for Mode (1,1) and a lower displacement at the centre than at the edges for Modes (1,3) and (1,5). The double joists at the unsupported edges clearly lowered the relative displacements at these locations in most cases, and the mode shapes approached those of four-side supported floors for the first two modes. While mode shape (1,3) of Floor 2 N was rather inconclusive, the one of Floor 1 N showed that the movement at the edges was relatively lowered.

Table 5.29: Natural frequencies due to the use of double joists or stiffer single joist

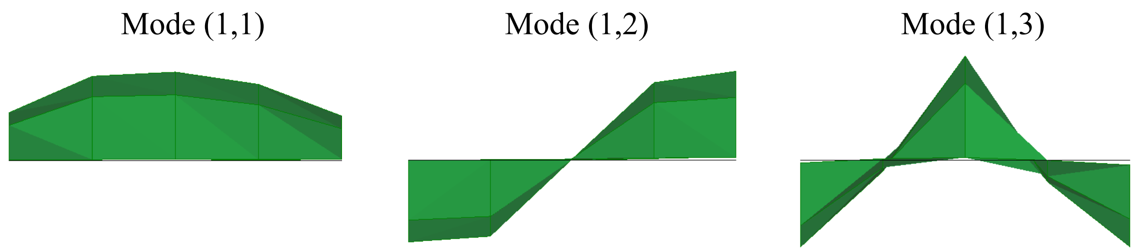
Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	24.46	30.11	35.58	47.33	60.72
1 D	25.23	30.99	36.45	48.53	59.34
Variation [%]	**+ 3.15**	**+ 2.92**	**+ 2.45**	**+ 2.54**	**- 2.27**
1 J	25.43	30.26	36.86	51.55	60.39
Variation [%]	**+ 3.97**	**+ 0.50**	**+ 3.60**	**+ 8.92**	**- 0.54**
1 G	27.29	30.84	39.17	51.38	60.63
Variation [%]	**+ 11.57**	**+ 2.42**	**+ 10.09**	**+ 8.56**	**- 0.15**
1 M	24.66	32.26	37.28	50.36	61.16
Variation [%]	**+ 0.82**	**+ 7.14**	**+ 4.78**	**+ 6.40**	**+ 0.72**
1 N	26.97	33.25	37.93	55.06	62.49
Variation [%]	**+ 10.26**	**+ 10.43**	**+ 6.60**	**+ 16.33**	**+ 2.92**
1 B	16.26	17.69	19.25	20.53	-
1 E	16.62	17.62	19.60	22.74	-
Variation [%]	**+ 2.21**	**- 0.40**	**+ 1.82**	**+ 10.76**	
1 K	16.76	17.49	19.78	-	-
Variation [%]	**+ 3.08**	**- 1.13**	**+ 2.75**		
1 H	17.37	17.89	21.86	22.68	-
Variation [%]	**+ 6.83**	**+ 1.13**	**+ 13.56**	**+ 10.47**	
1 C	13.48	14.57	16.69	18.67	-
1 F	13.62	14.25	16.29	18.53	-
Variation [%]	**+ 1.04**	**- 2.20**	**- 2.40**	**- 0.75**	
1 L	13.85	14.30	16.78	18.77	-
Variation [%]	**+ 2.74**	**- 1.85**	**+ 0.54**	**+ 0.54**	
1 I	14.08	14.73	18.74	19.59	-
Variation [%]	**+ 4.45**	**+ 1.10**	**+ 12.28**	**+ 4.93**	
2 A	22.66	28.81	34.88	47.65	59.63
2 J	23.87	28.58	35.34	49.79	58.85
Variation [%]	**+ 5.34**	**- 0.80**	**+ 1.32**	**+ 4.49**	**- 1.31**
2 G	24.89	28.57	40.17	45.51	57.81
Variation [%]	**+ 9.84**	**- 0.83**	**+ 15.17**	**- 4.49**	**- 3.05**
2 M	22.77	31.49	36.97	48.99	61.33
Variation [%]	**+ 0.49**	**+ 9.30**	**+ 5.99**	**+ 2.81**	**+ 2.85**
2 N	23.98	31.57	38.22	51.84	61.99
Variation [%]	**+ 5.83**	**+ 9.58**	**+ 9.58**	**+ 8.79**	**+ 3.96**

Table 5.30: Damping ratios due to the use of double joists or stiffer single joist

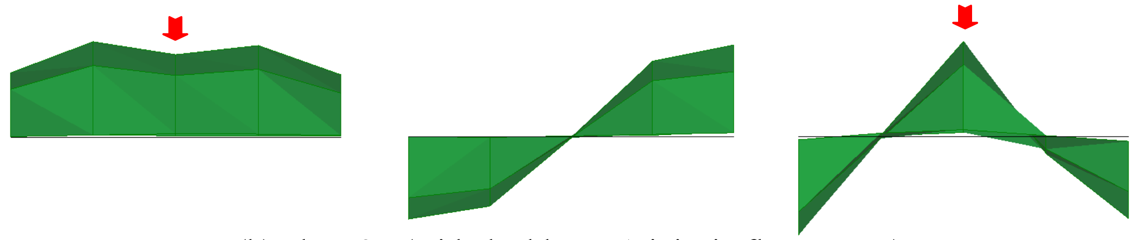
Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
1 A	3.98	0.95	1.08	1.13	1.16
1 D	2.51	1.64	0.96	0.98	1.04
Variation [%]	**- 36.93**	**+ 72.63**	**- 11.11**	**- 13.27**	**- 10.34**
1 J	3.56	1.00	0.95	1.64	1.02
Variation [%]	**- 10.55**	**+ 5.26**	**- 12.04**	**+ 45.13**	**- 12.07**
1 G	3.98	1.03	1.04	1.13	1.10
Variation [%]	**0.00**	**+ 8.42**	**- 3.70**	**0.00**	**- 5.17**
1 M	3.32	1.70	1.96	1.07	0.97
Variation [%]	**- 16.58**	**+ 78.95**	**+ 81.48**	**- 5.31**	**- 16.38**
1 N	2.80	1.04	1.28	1.16	0.94
Variation [%]	**- 29.65**	**+ 9.47**	**+ 18.52**	**+ 2.65**	**- 18.97**
1 B	1.57	1.05	2.39	2.22	-
1 E	1.37	1.59	1.53	2.22	-
Variation [%]	**- 12.74**	**+ 51.43**	**- 35.98**	**0.00**	
1 K	1.48	1.14	1.00	-	-
Variation [%]	**- 5.73**	**+ 8.57**	**- 58.16**		
1 H	1.25	1.38	1.37	3.05	-
Variation [%]	**- 20.38**	**+ 31.43**	**- 42.68**	**+ 37.39**	
1 C	1.09	1.52	3.25	3.28	-
1 F	1.26	1.12	4.18	2.31	-
Variation [%]	**+ 15.60**	**- 26.32**	**+ 28.62**	**- 29.57**	
1 L	1.11	1.18	3.56	2.84	-
Variation [%]	**+ 1.83**	**- 22.37**	**+ 9.54**	**- 13.41**	
1 I	1.43	1.37	3.76	3.02	-
Variation [%]	**+ 31.19**	**- 9.87**	**+ 15.69**	**- 7.93**	
2 A	2.41	0.99	0.80	1.10	1.02
2 J	2.71	0.97	0.99	1.19	1.03
Variation [%]	**+ 12.45**	**- 2.02**	**+ 23.75**	**+ 8.18**	**+ 0.98**
2 G	3.20	1.02	1.13	0.90	1.34
Variation [%]	**+ 32.78**	**+ 3.03**	**+ 41.25**	**- 18.18**	**+ 31.37**
2 M	2.93	0.96	0.96	1.18	1.08
Variation [%]	**+ 21.58**	**- 3.03**	**+ 20.00**	**+ 7.27**	**+ 5.88**
2 N	2.74	0.85	1.74	1.15	1.06
Variation [%]	**+ 13.69**	**- 14.14**	**+ 117.50**	**+ 4.55**	**+ 3.92**

Table 5.31: Deflections due to the use of double joists or stiffer single joist

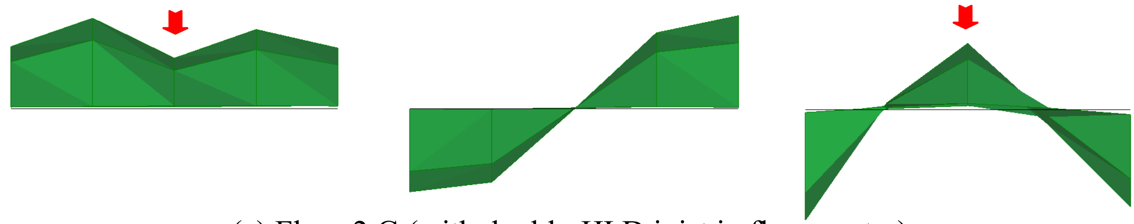
Floor	w_A [mm]	w_B [mm]
1 A	1.31	1.38
1 D	0.90	1.35
Variation [%]	**- 31.30**	**- 2.17**
1 J	0.72	1.32
Variation [%]	**- 45.04**	**- 4.35**
1 G	0.42	1.24
Variation [%]	**- 67.94**	**- 10.14**
1 M	1.37	1.34
Variation [%]	**+ 4.58**	**- 2.90**
1 N	0.71	1.22
Variation [%]	**- 45.80**	**- 11.59**
1 B	1.29	-
1 E	0.92	-
Variation [%]	**- 28.68**	
1 K	0.73	-
Variation [%]	**- 43.41**	
1 H	0.43	-
Variation [%]	**- 66.67**	
1 C	1.29	-
1 F	0.93	-
Variation [%]	**- 27.91**	
1 L	0.71	-
Variation [%]	**- 44.96**	
1 I	0.43	-
Variation [%]	**- 66.67**	
2 A	1.59	1.68
2 J	0.85	1.35
Variation [%]	**- 46.54**	**- 19.64**
2 G	0.56	1.47
Variation [%]	**- 64.80**	**- 12.50**
2 M	1.57	1.38
Variation [%]	**- 1.26**	**- 17.86**
2 N	0.80	1.33
Variation [%]	**- 49.41**	**- 20.83**



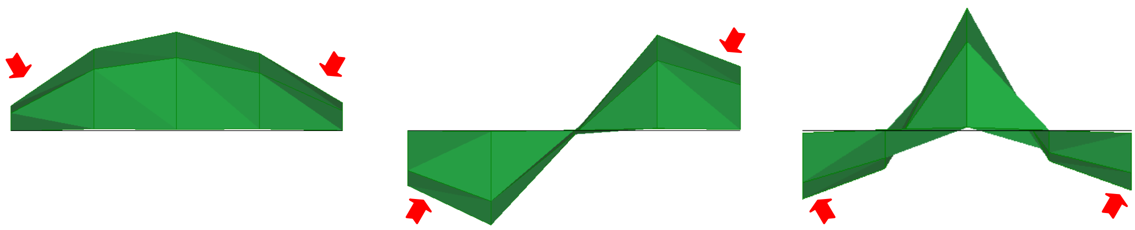
(a) Floor 2 A (with single JJI A joists)



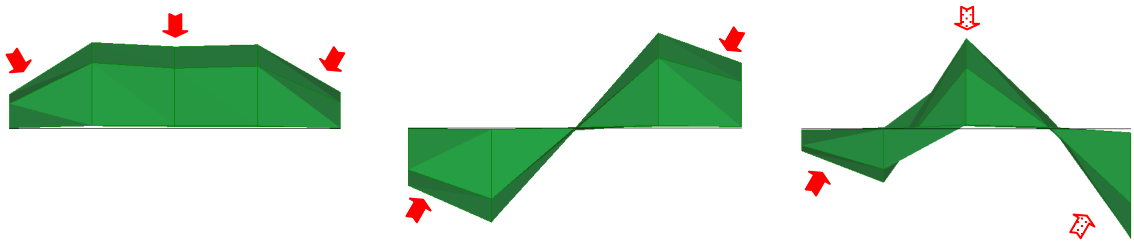
(b) Floor 2 J (with double JJI A joist in floor centre)



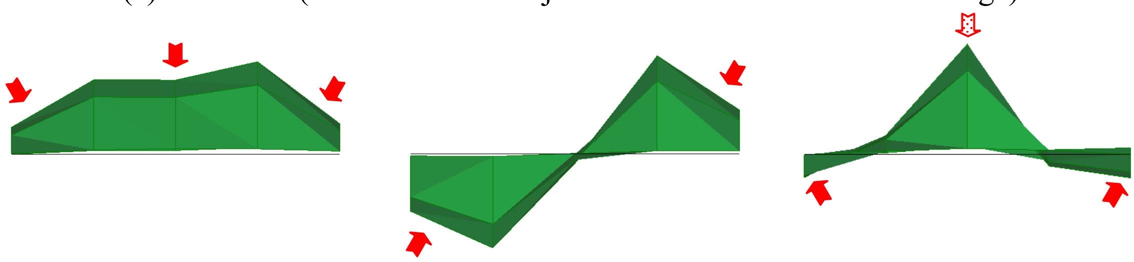
(c) Floor 2 G (with double JJI D joist in floor centre)



(d) Floor 2 M (with double JJI A joist at each edge)



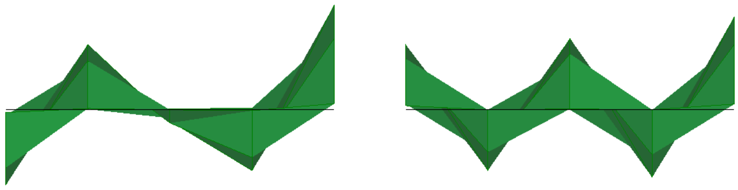
(e) Floor 2 N (with double JJI A joist in floor centre and at each edge)



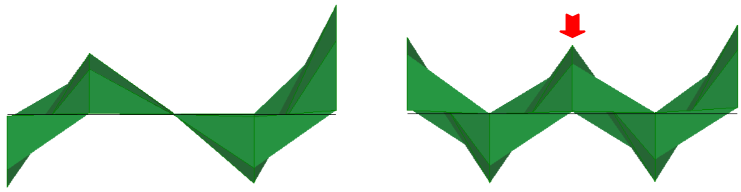
(f) Floor 1 N (with double JJI A joists in floor centre and at each edge)

Figure 5.10: Principal mode shapes of Modes (1,1) to (1,3) in transverse direction

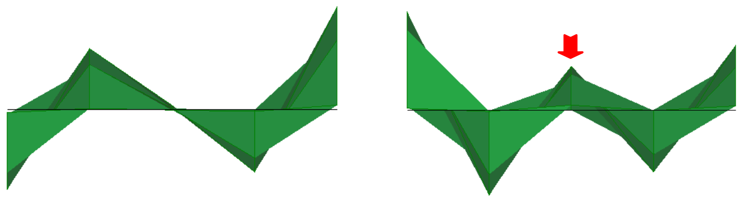
Mode (1,4) Mode (1,5)



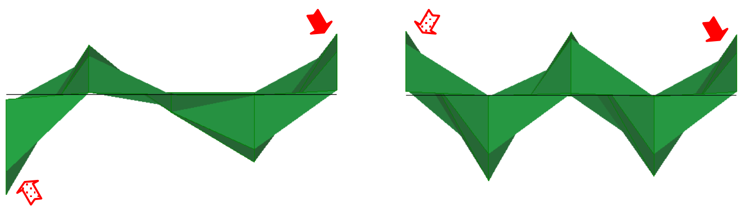
(a) Floor 2 A (with single JJI A joists)



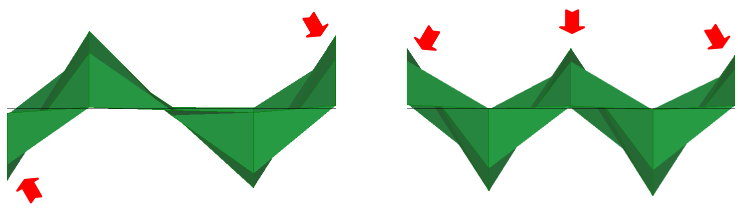
(b) Floor 2 J (with double JJI A joist in floor centre)



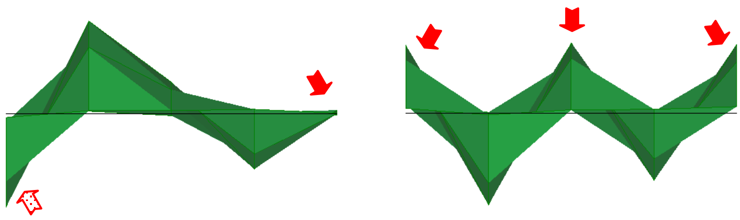
(c) Floor 2 G (with double JJI D joist in floor centre)



(d) Floor 2 M (with double JJI A joist at each edge)



(e) Floor 2 N (with double JJI A joist in floor centre and at each edge)



(f) Floor 1 N (with double JJI A joists in floor centre and at each edge)

Figure 5.11: Principal mode shapes of Modes (1,4) and (1,5) in transverse direction

Using a stiffer single joist or double joists along the longitudinal floor central line increased the frequencies of Modes (1,1) and (1,3) of the tested floors, except the one of Mode (1,3) of Floor 1 F. The effect of replacing the single JJI A joist in floor centre by the stiffer single JJI D joist was mainly insignificant. Further replacement by double JJI A joists for this location was more effective with an increase in frequencies by 2.74% to 5.34% for Mode (1,1) and a maximum of 3.60% for Mode (1,3). However, most effective regarding these modes was the use of the double JJI D joists in floor centre with an increase of double percentage figures in the natural frequencies corresponding to Mode (1,3) with a maximum of 15.17%. For the same measure, the fundamental frequency increased by 11.57% and 9.84% for the unloaded floors. In general, the efficiency of using a stiffer single joist or double joists in floor centre became lower for the loaded floors compared to the unloaded ones. Using double joists of regular size only at the edges increased the frequencies of Mode (1,2) by up to 9.30%. Also the frequencies of the other modes were raised with the lowest effect on Mode (1,1). Using double joists of regular size at the edges and in floor centre notably raised the natural frequencies of Modes (1,1) to (1,4), in Test series 1 by more than 10%, apart from Mode (1,3) with 6.60%, and also raised the frequencies of Mode (1,5). The frequency spacing varied depending on the location where stiffer single joist or double joists were introduced and thus on the degree of the effect on the frequencies of the individual modes.

The variation of damping ratios when introducing double joists was inconclusive (Table 5.30). For the unloaded floors, all the damping ratios decreased for Mode (1,1) (with one exception) and Mode (1,5) and increased for Mode (1,2) in Test series 1, and exactly the opposite was noted for Test series 2 (with one exception for Mode (1,2)). For the loaded floors, one series showed completely the opposite phenomena on damping for all modes (with only one exception) than the other series.

Table 5.31 shows that double joists reduced the deflections significantly at the sensitive locations they were placed at and also the mid-span deflection of adjacent joists. The deflection at the floor centre sustained a largest reduction due to the configuration with double central JJI D joists by about 67% for floors in Test series 1 and 64.80% in Test series 2. The lowest variation of deflection at the floor centre was found for the floors with double joists at the edges. Whereas the variation in deflection at the floor centre was similar for comparable floor modifications, the deflection at mid-span of the adjacent joist illustrated higher efficiency in Test series 2. A discussion of these results can be found in Section 5.2.8.

5.2.6 Effect of imposed load

The addition of mass significantly lowered the natural frequencies of all flooring structures of this study (Table 5.32) and highly affected the damping ratios (Table 5.33).

Table 5.32: Natural frequencies due to raised floor mass

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
1 A	24.46	30.11	35.58	47.33	60.72
1 B	16.26	17.69	19.25	20.53	-
Variation [%]	**- 33.52**	**- 41.25**	**- 45.90**	**- 56.62**	
1 C	13.48	14.57	16.69	18.67	-
Variation [%]	**- 44.89**	**- 51.61**	**- 53.09**	**- 60.55**	
1 D	25.23	30.99	36.45	48.53	59.34
1 E	16.62	17.62	19.60	22.74	-
Variation [%]	**- 34.13**	**- 43.14**	**- 46.23**	**- 53.14**	
1 F	13.62	14.25	16.29	18.53	-
Variation [%]	**- 46.02**	**- 54.02**	**- 55.31**	**- 61.82**	
1 G	27.29	30.84	39.17	51.38	60.63
1 H	17.37	17.89	21.86	22.68	-
Variation [%]	**- 36.35**	**- 41.99**	**- 44.19**	**- 55.86**	
1 I	14.08	14.73	18.74	19.59	-
Variation [%]	**- 48.41**	**- 52.24**	**- 52.16**	**- 61.87**	
1 J	25.43	30.26	36.86	51.55	60.39
1 K	16.76	17.49	19.78	-	-
Variation [%]	**- 34.09**	**- 42.20**	**- 46.34**		
1 L	13.85	14.30	16.78	18.77	-
Variation [%]	**- 45.54**	**- 52.74**	**- 54.48**	**- 63.59**	
2 Q	22.11	27.55	33.82	47.04	57.52
2 R	14.68	15.99	18.39	19.85	-
Variation [%]	**- 33.60**	**- 41.96**	**- 45.62**	**- 57.80**	
2 S	12.16	13.19	15.32	19.32	-
Variation [%]	**- 45.00**	**- 52.12**	**- 54.70**	**- 58.93**	

The frequencies of Mode (1,1) were lowered by more than a third for the first increment in mass and almost halved for the second increment compared to the base floor. The degree of the effect was, however, descending. The effect usually became stronger for subsequent modes. In general, the degree of the effect was usually about the same for individual natural frequencies independent from the structural detailing of the flooring structure. In the presented cases, the structural differences included joist arrangements (e.g. double joists), joist depth and support conditions. For the first increment in floor mass, the spacing between frequencies of two adjacent modes was considerably reduced, for Modes (1,3) and (1,4) from 11.75 Hz or above down to below 5 Hz. The frequency spacing between Modes (1,1) and (1,2) fell below 2 Hz, and for the floors with double joists in floor centre even below 1 Hz. For the following surcharge, the spacing was in general less affected. The very low spacing between Modes (1,1) and (1,2) led to notable co-action of these modes in the floors of Test series 1, particularly for those with the double joists in floor centre. This can be seen from the corresponding mode shapes as illustrated for Floor 1 I in Figure 5.12. Otherwise, the mode shapes of the loaded and unloaded floors were similar. These effects are further discussed in Section 5.2.8.

Table 5.33: Damping ratios due to raised floor mass

Floor	$\zeta_{(1,1)}$ **[%]**	$\zeta_{(1,2)}$ **[%]**	$\zeta_{(1,3)}$ **[%]**	$\zeta_{(1,4)}$ **[%]**	$\zeta_{(1,5)}$ **[%]**
1 A	3.98	0.95	1.08	1.13	1.16
1 B	1.57	1.05	2.39	2.22	-
Variation [%]	**- 60.55**	**+ 10.53**	**+ 121.30**	**+ 96.46**	
1 C	1.09	1.52	3.25	3.28	-
Variation [%]	**- 72.61**	**+ 60.00**	**+ 200.93**	**+ 190.27**	
1 D	2.51	1.64	0.96	0.98	1.04
1 E	1.37	1.59	1.53	2.22	-
Variation [%]	**- 45.42**	**- 3.05**	**+ 59.38**	**+ 126.53**	
1 F	1.26	1.12	4.18	2.31	-
Variation [%]	**- 49.80**	**- 31.71**	**+ 335.42**	**+ 135.71**	
1 G	3.98	1.03	1.04	1.13	1.10
1 H	1.25	1.38	1.37	3.05	-
Variation [%]	**- 68.59**	**+ 33.98**	**+ 31.73**	**+ 169.91**	
1 I	1.43	1.37	3.76	3.02	-
Variation [%]	**- 64.07**	**+ 33.01**	**+ 261.54**	**+ 167.26**	
1 J	3.56	1.00	0.95	1.64	1.02
1 K	1.48	1.14	1.00	-	-
Variation [%]	**- 58.43**	**+ 14.00**	**+ 5.26**		
1 L	1.11	1.18	3.56	2.84	-
Variation [%]	**- 68.82**	**+ 18.00**	**+ 274.74**	**+ 73.17**	
2 Q	2.79	1.04	0.94	1.06	0.86
2 R	1.31	0.98	1.34	1.25	-
Variation [%]	**- 53.05**	**- 5.77**	**+ 42.55**	**+ 17.92**	
2 S	1.25	1.06	1.24	3.08	-
Variation [%]	**- 55.20**	**+ 1.92**	**+ 31.91**	**+ 190.57**	

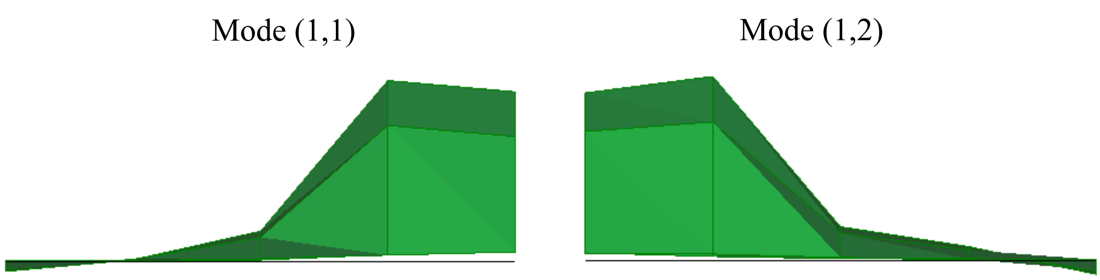


Figure 5.12: Shapes of Modes (1,1) and (1,2) of Floor 1 I

The damping ratios corresponding to the fundamental mode of vibration were highly reduced by between 45.42% and 72.61% (Table 5.33). The effect was usually raised for further increase in mass, but the degree of the effect became lowered. The variation of the damping ratios of Mode (1,2) was unsteady, sometimes decreasing but most of the time increasing. For the damping ratios corresponding to Modes (1,3) and (1,4) a clear increase was found, often enormous with triple percentage figures, with a higher significance for further surcharging the floor. The effect on damping is further analysed in Chapter 8.

The deflection of the floors usually varied insignificantly as can be seen from Table 5.34.

Table 5.34: Deflections at floor centre due to raised floor mass

Floor	w_A **[mm]**
1 A	1.31
1 B	1.29
Variation [%]	**- 1.53**
1 C	1.29
Variation [%]	**- 1.53**
1 D	0.90
1 E	0.92
Variation [%]	**+ 2.22**
1 F	0.93
Variation [%]	**+ 3.33**
1 G	0.42
1 H	0.43
Variation [%]	**+ 2.38**
1 I	0.43
Variation [%]	**+ 2.38**
1 J	0.72
1 K	0.73
Variation [%]	**+ 1.39**
1 L	0.71
Variation [%]	**- 1.39**
2 Q	1.58
2 R	-
Variation [%]	
2 S	1.51
Variation [%]	**- 4.43**

5.2.7 Effect of reusing the material

The effect due to reusing floor material on natural frequencies and damping ratios was very small. From Table 5.35, the frequencies decreased slightly for most of the modes, with a maximum of 3.74% for Mode (1,4). The damping ratios increased for the first three principal modes and decreased for the other two investigated (Table 5.36). The absolute values of the damping ratios were rather similar for the same mode of the two tests and only the one for Mode (1,3) varied in a noteworthy way.

Table 5.35: Natural frequencies due to reusing materials

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
2 A	22.66	28.81	34.88	47.65	59.63
2 A**	22.49	28.58	33.98	45.87	58.27
Variation [%]	**- 0.75**	**- 0.80**	**- 2.58**	**- 3.74**	**- 2.28**

Table 5.36: Damping ratios due to reusing materials

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]
2 A	2.41	0.99	0.80	1.10	1.02
2 A**	2.64	1.04	1.12	0.91	0.96
Variation [%]	**+ 9.54**	**+ 5.05**	**+ 40.00**	**- 17.27**	**- 5.88**

5.2.8 Discussion on the results of JJI floor test series

Flooring systems with JJI-joists were investigated for varying structural and non-structural modifications, with focus on the effect of the use of double joists and raised floor mass.

The deflection was measured at the floor centre but also at the mid-span location of other joists. The deflection at locations of joists under load adjacent to the centre or edges was investigated to explore the efficiency of structural modifications on floor areas other than the central one. In case of Floor 1 A, the deflection was even measured at one of the unsupported edges (Figure 4.17(a)). Guidelines, such as EC5-1-1, refer to the maximum deflection under load applied at any point on the floor. If floor edges are unsupported, highest deflection can be expected at those locations. The results showed that the deflection usually increased if the load was close to one of the unsupported edges and that the deflection indeed was highest at the free edge. Nevertheless, the attention was focused on deflection at the centre of the floors in this test series.

Reusing the floor material and screw holes was affecting the natural frequencies or damping ratios to a relatively low degree, and therefore the level of measurement uncertainty due to the reused material was rather acceptable.

From researching the effects of structural modifications, it could be found that although a different method was applied to connect the floor to the supports compared to the TJI floor test series, the effect and efficiency was rather the same and thus insignificant.

Both measures of reducing the joist spacing or raising the joist depth increase the stiffness especially in span direction and therefore lower the degree of isotropy, which should reduce the spacing between natural frequencies. This was usually true for the tested floors with deeper joists whereas the spacing between Modes (1,3) and (1,4) increased for the floors with double joists. Reducing the joist spacing by a third or by a half was usually more effective on frequencies and deflection than raising the joist depth from 220 mm to 245 mm, up by 11.36%. To gain stronger effects for the latter case, the joist depth would need to be further raised, which would generally be influential on constructional dimensions. Reduced joist spacing could be applied without large complications, and it may be more beneficial regarding local deflection of the decking board at areas between joists. Its efficiency of clearly lowering the deflections was higher on the floors with lower joist depth. The increase in damping mainly found for Modes (1,1) and (1,2) on floors with deeper joists may form an argument for preference of those.

The use of the I-joist blocking raised the degree of isotropy and hence a raised spacing between neighbouring frequencies could be expected. The frequencies of Modes (1,1) and (1,2) were not affected. The natural frequencies of higher modes clearly increased, with higher efficiency for an increasing number of blocking rows, whereas only the frequencies of Modes (1,2) and (1,3) were also considerably separated. This did not fully agree with the theory. However, Hu and Tardif (2000) found little effect of I-joist blocking on any of the natural frequencies corresponding to Modes (1,1) to (1,5) of their test floors. From comparison of the mode shapes, the movement at floor centre for Mode (1,3) became fairly similar or less influential with respect to the one at the edges when I-joist blocking was used. The movement at the edges then became more significant for all of the first three principal modes. The damping ratios were fairly similar for individual modes, which indicates that the addition of I-joist blocking did not influence the damping. The deflections at the floor centre and at mid-span of the adjacent joists were reduced by about a quarter or more since the installation of the blocking elements enhanced the distribution of the load to adjacent joists, which means an increased load sharing effect.

The only non-structural modification of the test series investigated was the raise of floor mass without altering the stiffness. The mass was largely increased up to about 76 kg/m^2 depending on the structural details, which may make the modified floors no longer light-weight. The mass was observed to significantly influence the natural frequencies and damping ratios, not only severely reducing the natural frequencies of all identified modes but also largely narrowing the spacing between them so as to become critical, which can result in enlarged co-action of the modes.

Stronger interaction of modes may produce relatively high amplitudes and should be avoided. As the distance between the natural frequencies of the loaded floors was extremely low, a considerable increase in floor stiffness would be required to compensate this mass effect. A raised stiffness in transverse direction would more effectively increase the frequencies of higher modes. To balance the effect of lowered fundamental frequency due to higher mass, an increase in stiffness in longitudinal direction would rather be required. For heavier floors attempts should therefore be made to at least raise the transverse stiffness of the floor so as to extend the spacing between adjacent natural frequencies.

As described in Section 4.2.2, the stiffness of the central joist and the movement at unrestrained edges considerably contribute to unsatisfactory vibration. The application of double joists for effectively stiffening these sensitive locations and raising the modal mass at anti-nodes of the most influential modes worked well for the light-weight floors tested due to manipulation of the lower and partially higher modes. As the modal mass was raised, the initial peak velocities were not expected to increase.

The effect of double joists of regular size on natural frequencies was rather small if they were only used in floor centre but this effect could be significant when the cross sections of the joists were increased or the floor edges stiffened additionally. With increasing mode number, the number of anti-nodes also raises. To yield notable effects on the natural frequencies of higher modes, more anti-node locations should be stiffened, which can be noticed from the effect on Mode (1,5). Its frequency rose when at least two locations with high vibration displacements were stiffened.

Stiffening only the floor centre may not be beneficial for the floors with a critical spacing between adjacent natural frequencies. This measure would raise the fundamental frequency but rather not the one of the successive mode so as to intensify the interaction of these modes. The use of double joists indeed raises the degree of anisotropy. The targeted application of the joists showed, however, that it is possible to mainly increase the distances between the frequencies of Modes (1,1) to (1,4) if edges and floor centre were stiffened.

Stiffening of anti-node locations by double joists can lead to lowest movement at the centre of the whole cross section for Mode (1,1), and also to relatively low movement at the edges of two-side supported floors. For the latter the mode shapes approach those for four-side supported floors, at least for Modes (1,1) and (1,2).

The deflection was considerably lowered locally at the position of the double joists, and the deflections on the neighbouring joists were also decreased since smaller load was transferred. Deflection at the edges was not measured, but a similar reduction trend to the one observed for the application of double joists in floor centre could be expected.

Even though an increase in natural frequencies of the lower modes and a decrease in static deflections may also be achieved by lowering the joist spacing, double joists could more effectively stiffen the most sensitive locations and raise the frequency spacing of lower modes when they are used deliberately. As the use of blocking did not affect the first two vibration modes but higher ones, this configuration may be used in conjunction with double joists to raise all of the natural frequencies without lowering the frequency spacing. This would further enhance the efficiency of lowering the deflection due to the increase in the load sharing effect.

6. Results of the metal-web joist floor test series

The MWJ test floors used were marginal in design, and some were rather safe but some less, depending on the structural configuration. Besides investigating the variations in dynamic parameters due to structural modifications, the test results were compared to predictions based on the design criteria of EC5-1-1 and the UK NA to EC5-1-1. The calculations of the velocity response and design limit were then repeated using the damping ratio corresponding to the fundamental vibration mode, the number of first order modes below 40 Hz and the fundamental frequency obtained from the experimental investigations, and used for the comparison. Therefore, the presented results include all first order modes below 40 Hz, which are summarised in Table 6.1 and Table 6.2. Some natural frequencies of first order modes above 40 Hz are included in Table 6.1 as those will be used for comparisons. The absolute variation in damping was rather insignificant (Table 6.2). The damping was further considered in Section 6.5 and Chapter 8.

Table 6.1: Natural frequencies of first order modes below 40 Hz of all MWJ floors

Floor	$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]]	$f_{(1,5)}$ [Hz]	$f_{(1,6)}$ [Hz]	$f_{(1,7)}$ [Hz]	$f_{(1,8)}$ [Hz]
A	14.42	16.01	18.07	20.60	23.71	27.28	31.34	35.26
B	14.59	16.23	21.22	28.40	37.09	44.56	-	-
C	14.55	16.21	21.27	28.27	37.08	-	-	-
D	14.49	16.18	20.09	25.27	33.81	42.82	-	-
E	14.53	16.21	19.63	24.10	30.47	37.24	-	-
F	13.43	15.52	19.78	25.12	32.17	39.35		-
G	15.54	16.99	18.80	20.97	23.77	27.37	30.92	34.23
H	15.47	17.02	19.87	23.97	30.04	36.72	44.06	-
I	14.25	16.16	19.74	24.69	31.69	39.13	-	-

Table 6.2: Damping ratios of first order modes below 40 Hz of all MWJ floors

Floor	$\zeta_{(1,1)}$ [%]	$\zeta_{(1,2)}$ [%]	$\zeta_{(1,3)}$ [%]	$\zeta_{(1,4)}$ [%]	$\zeta_{(1,5)}$ [%]	$\zeta_{(1,6)}$ [%]	$\zeta_{(1,7)}$ [%]	$\zeta_{(1,8)}$ [%]
A	0.88	0.95	0.79	0.88	1.03	0.85	1.18	1.20
B	0.83	0.70	1.08	1.03	1.15	-	-	-
C	0.88	0.77	0.91	1.34	1.30	-	-	-
D	0.87	0.80	0.85	1.15	1.29	-	-	-
E	0.79	0.83	0.90	1.43	1.37	1.40	-	-
F	0.99	0.85	0.83	1.38	1.59	1.28	-	-
G	0.77	1.05	1.13	1.39	1.46	1.24	1.17	1.05
H	0.94	0.73	1.01	1.11	0.96	1.22	-	-
I	0.80	0.73	0.98	1.19	1.36	1.18	-	-
Mean	0.86	0.82	0.94	1.21	1.28	1.19	1.18	1.12

The modes shapes in transverse direction of Modes (1,1) to (1,3) of all test floors are shown in Figure 6.1 as some variations due to the structural modification by adding ceiling and utilising strongback occurred. The shapes of higher modes are not included since distinct variations in those shapes were not identified.

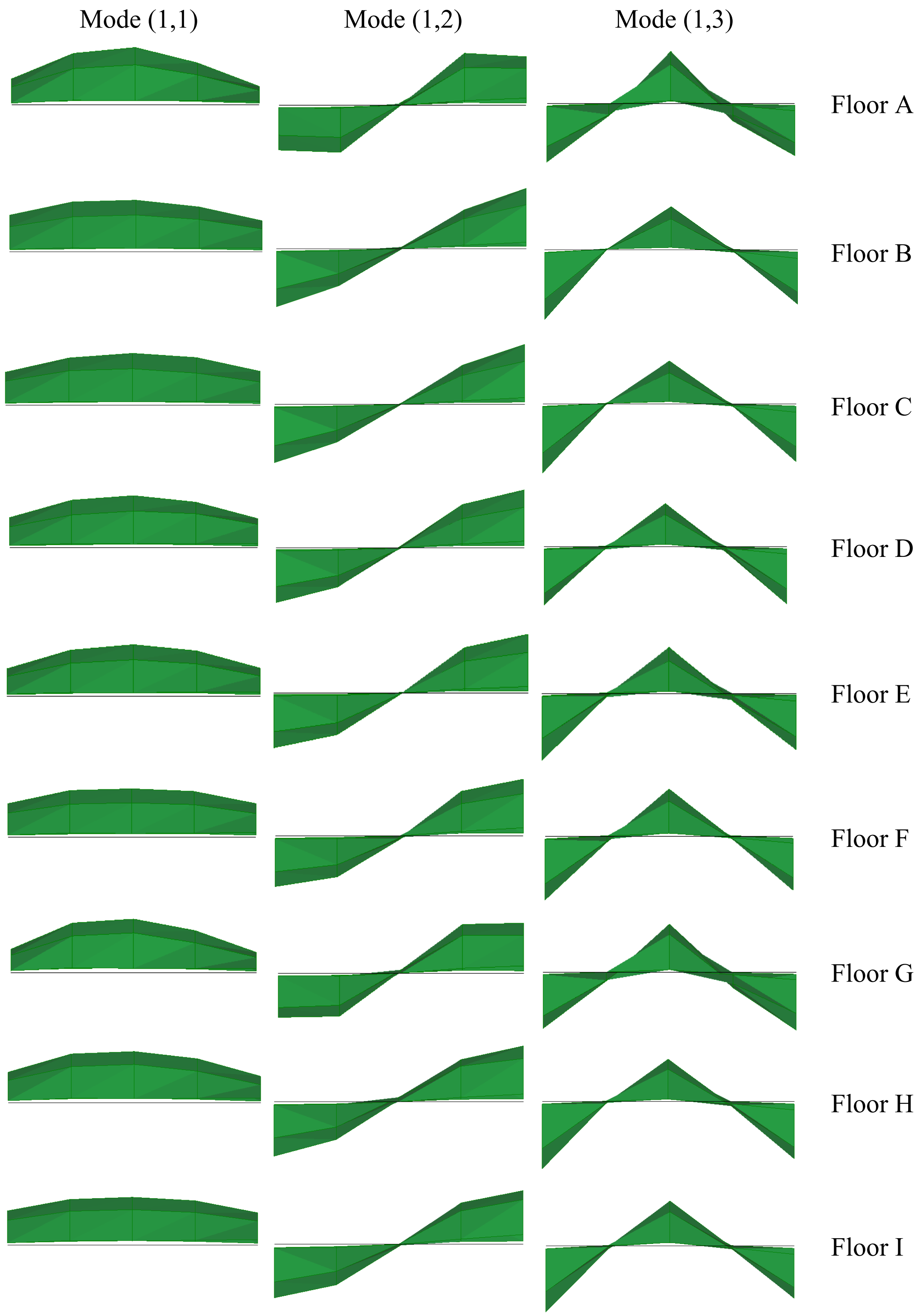


Figure 6.1: Mode shapes of principal Modes (1,1) to (1,3)

Table 6.3 and Table 6.4 present the net mid-span deflections under varying load locations of Floor A and G respectively, which represent the base floors with 600 and 400 mm joist spacing. The maximum deflection was observed at the loading point at mid-span of the unsupported edges. All other mid-span deflections were similar to each other for individual floors. The point load deflection at the floor centre was selected as the reference point for comparisons of all floors (Figure 6.2).

Table 6.3: Net mid-span deflection of all joists for all load cases of Floor A

Joist loaded	**Net mid-span deflection *w* of all joists [mm]**								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1	2.48	0.69	0.10	0.01	0.00	0.00	0.00	-0.01	-0.04
2	0.68	1.68	0.62	0.12	0.02	0.01	0.00	-0.01	-0.02
3	0.10	0.67	1.73	0.62	0.06	0.02	0.01	0.00	-0.02
4	-0.05	0.12	0.66	1.75	0.54	0.13	0.02	0.00	-0.01
5	-0.01	0.02	0.11	0.72	**1.80**	0.66	0.11	0.01	-0.03
6	-0.01	0.00	0.01	0.14	0.73	1.79	0.70	0.13	-0.03
7	-0.01	0.00	0.01	0.02	0.13	0.70	1.81	0.74	0.08
8	-0.03	-0.01	0.00	0.01	0.03	0.14	0.65	1.71	0.64
9	-0.05	-0.02	-0.01	0.00	0.00	0.00	0.06	0.59	2.50
Max.	2.48	1.68	1.73	1.75	**1.80**	1.79	1.81	1.71	2.50

Table 6.4: Net mid-span deflection of all joists for all load cases of Floor G

Joist loaded	**Net mid-span *w* deflection of joists [mm]**												
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
1	2.51	0.88	0.25	0.05	0.01	0.01	0.00	0.00	0.00	-0.01	-0.01	-0.01	-0.02
2	0.86	1.37	0.75	0.20	0.07	0.02	0.01	0.00	0.01	0.00	-0.01	-0.01	-0.02
3	0.21	0.81	1.27	0.64	0.23	0.05	0.02	0.01	0.01	0.00	0.00	0.00	-0.01
4	0.06	0.23	0.68	1.37	0.71	0.19	0.05	0.02	0.01	0.00	0.00	0.00	0.00
5	0.04	0.07	0.19	0.73	1.29	0.65	0.21	0.08	0.03	0.00	0.00	0.00	0.00
6	0.02	0.03	0.04	0.21	0.72	1.34	0.71	0.29	0.10	0.01	0.01	0.01	0.00
7	0.00	0.02	0.01	0.03	0.20	0.80	**1.44**	0.81	0.28	0.04	0.02	0.00	-0.01
8	0.01	0.01	0.00	0.01	0.06	0.31	0.75	1.39	0.80	0.24	0.07	0.02	-0.01
9	-0.01	0.01	0.01	0.01	0.02	0.08	0.21	0.80	1.28	0.77	0.28	0.07	0.01
10	0.00	0.01	0.00	0.00	0.01	0.02	0.04	0.21	0.74	1.46	0.88	0.26	0.02
11	0.01	0.01	0.00	0.00	0.00	0.01	0.01	0.05	0.20	0.67	1.48	0.86	0.24
12	-0.02	0.00	-0.01	0.00	0.00	0.01	0.01	0.01	0.01	0.18	0.84	1.47	0.99
13	-0.02	-0.01	-0.01	0.00	0.00	0.00	0.00	0.00	-0.01	-0.02	0.12	0.84	2.40
Max.	2.51	1.37	1.27	1.37	1.29	1.34	**1.44**	1.39	1.28	1.46	1.48	1.47	2.40

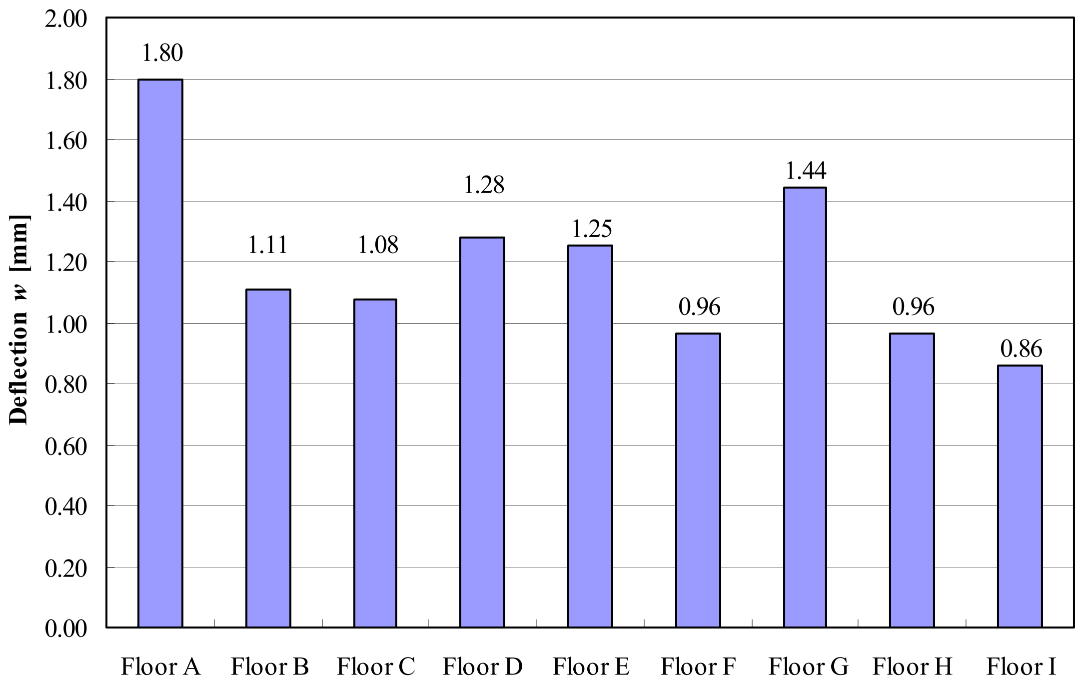
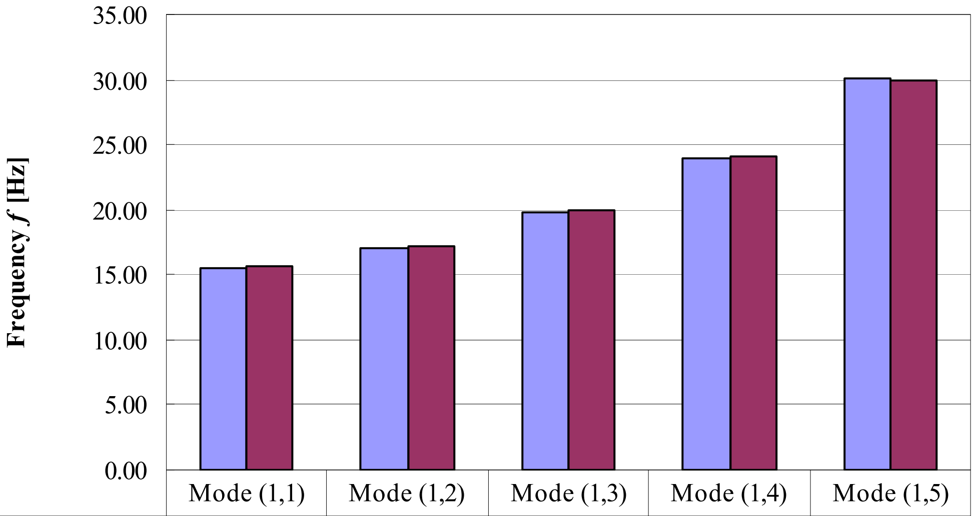


Figure 6.2: Unit point load deflections of all floors when loaded at the floor centre

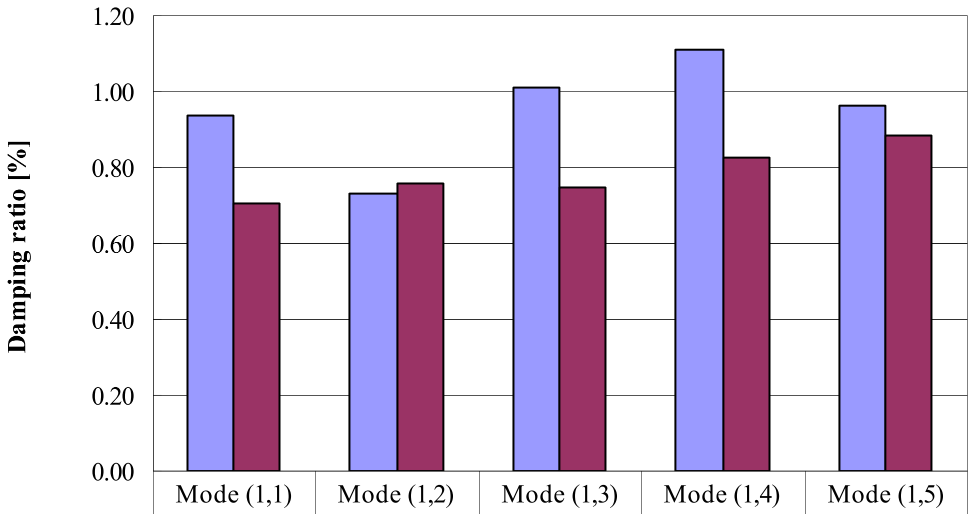
For these flooring tests the excitation method was varied compared to the tests carried out before. As the floor supports possessed a relatively large height, and this complicated the way to excite the whole top floor surface by a brush, a trolley with a long handle was used for excitation instead (Section 4.5). However, one floor test (MWJ Floor H) was repeated using the brush excitation method to investigate whether the varied excitation had a significant influence on the results. Figure 6.3 shows the natural frequencies and damping ratios for Modes (1,1) to (1,5) from the different excitation tests.

The natural frequencies were slightly lower in case of trolley excitation, and this was possibly because the introduction of the trolley slightly added weight to the floor. However, the variation in frequency was below 1% and can be neglected. The damping ratios were slightly higher in case of trolley excitation by an average of 0.28%, except the one for Mode (1,2). In general, the damping ratios for both excitation methods on the floors with metal-web joists were similar, and much lower than those on the I-joist floors, which indicates that lower damping for the floors with metal-web joists was not caused by the varied excitation method.



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)
Trolley excitation	15.47	17.02	19.87	23.97	30.04
Brush excitation	15.61	17.14	20.03	24.08	30.01

(a) Natural frequencies



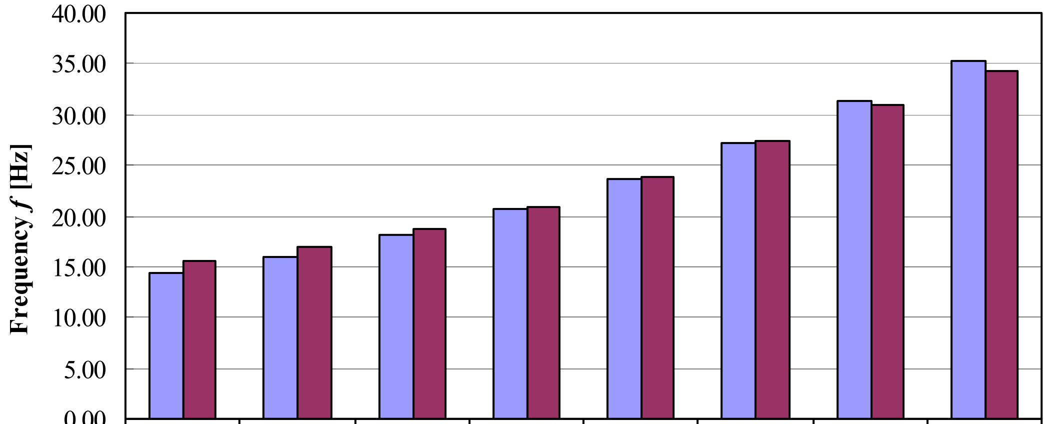
	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)
Trolley excitation	0.94	0.73	1.01	1.11	0.96
Brush excitation	0.71	0.76	0.75	0.83	0.88

(b) Damping ratios

Figure 6.3: Natural frequencies and damping ratios due to different excitation methods applied on Floor H

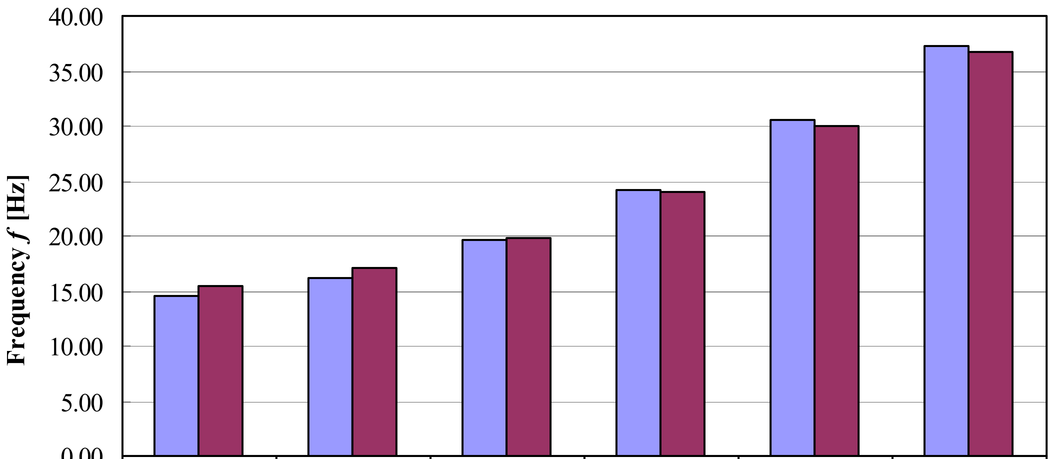
6.1 Effect of varied joist spacing

The identified natural frequencies of Floors A and G, Floors E and H and Floors F and I are illustrated in Figure 6.4 to reflect the impact of reducing the joist spacing from 600 mm to 400 mm. Figure 6.4(a) shows an increase in the compared frequencies for Modes (1,1) to (1,6), by 7.77% for the fundamental frequency and 4.04% for the third natural frequency whereas Modes (1,4) to (1,6) were hardly affected. The frequencies of Mode (1,7) and (1,8) were decreasing. Figure 6.4(b) and (c) show both moderate increase in the first two natural frequencies, with an increase of 6.47% for Floor H and 6.11% for Floor I for the fundamental mode, little effect on the third frequency and slight decrease in frequencies thereafter. The frequency spacing thus mainly decreased. The point from increasing to decreasing frequencies occurred earlier (at lower modes) for the floors with strongback and even earlier if ceiling was installed. The varying effects on lower and higher natural frequencies are caused by the varying influences of raised stiffness and mass due to the addition of joists. A reduced spacing between adjacent frequencies could be expected as the reduction of joist spacing raises the degree of orthotropy of the floor.



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)	Mode (1,7)	Mode (1,8)
Floor A	14.42	16.01	18.07	20.60	23.71	27.28	31.34	35.26
Floor G	15.54	16.99	18.80	20.97	23.77	27.37	30.92	34.23

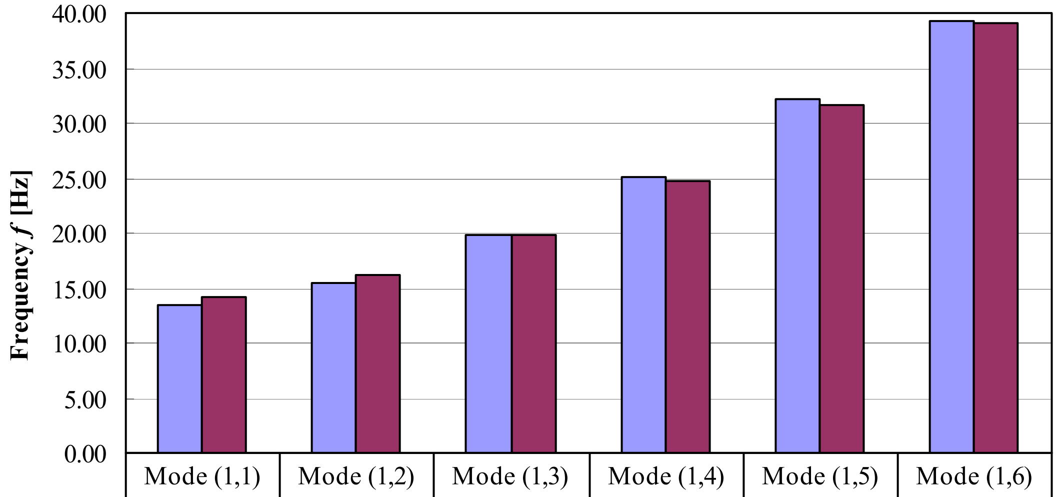
(a) Floors without strongback and ceiling



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor E	14.53	16.21	19.63	24.10	30.47	37.24
Floor H	15.47	17.02	19.87	23.97	30.04	36.72

(b) Floors with TR26 strongback of 35 mm × 97 mm at mid-span but without ceiling

Figure 6.4: Natural frequencies of the floors with 600 mm and 400 mm joist spacing



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor F	13.43	15.52	19.78	25.12	32.17	39.35
Floor I	14.25	16.16	19.74	24.69	31.69	39.13

(c) Floors with TR26 strongback of 35 mm × 97 mm at mid-span and ceiling

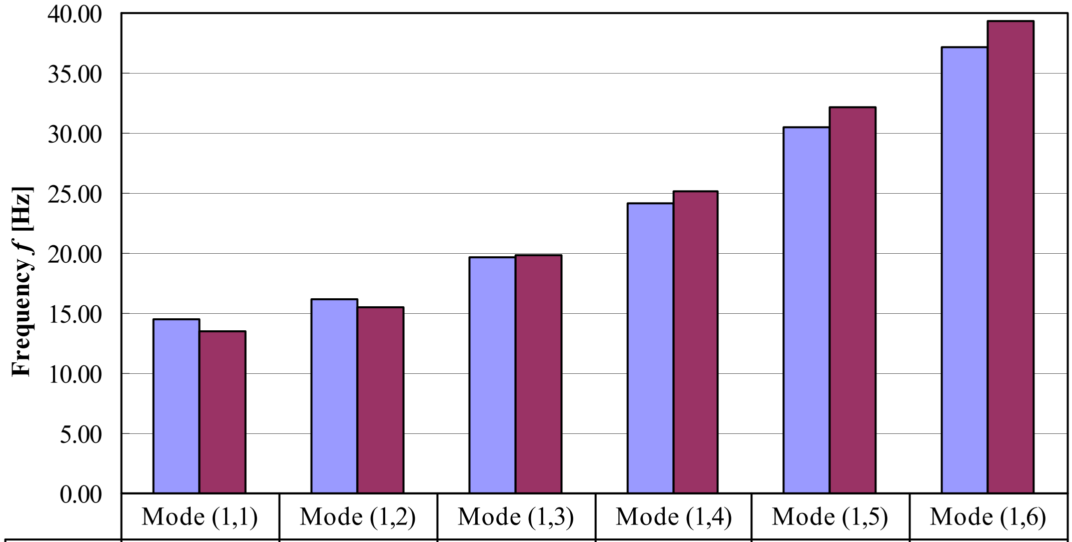
Figure 6.4: Natural frequencies of the floors with 600 mm and 400 mm joist spacing (cont.)

The variation of damping was rather inconclusive. The mode shapes of Mode (1,2) of Floors A and G (Figure 6.1) may imply that reducing the joist spacing shifted the anti-nodes to the edge. The other comparisons (Floors E and H, Floors F and I) instead indicate that the anti-node occurred at the edges independent from the joist spacing.

As can be seen from Figure 6.2, reducing the joist spacing could considerably lower the deflection, by 20.00% for Floors A and G and 23.20% for Floors E and H, all having no ceiling. A decrease of 11.34% was found for the floors with ceiling (Floors F and I).

6.2 Effect of ceiling

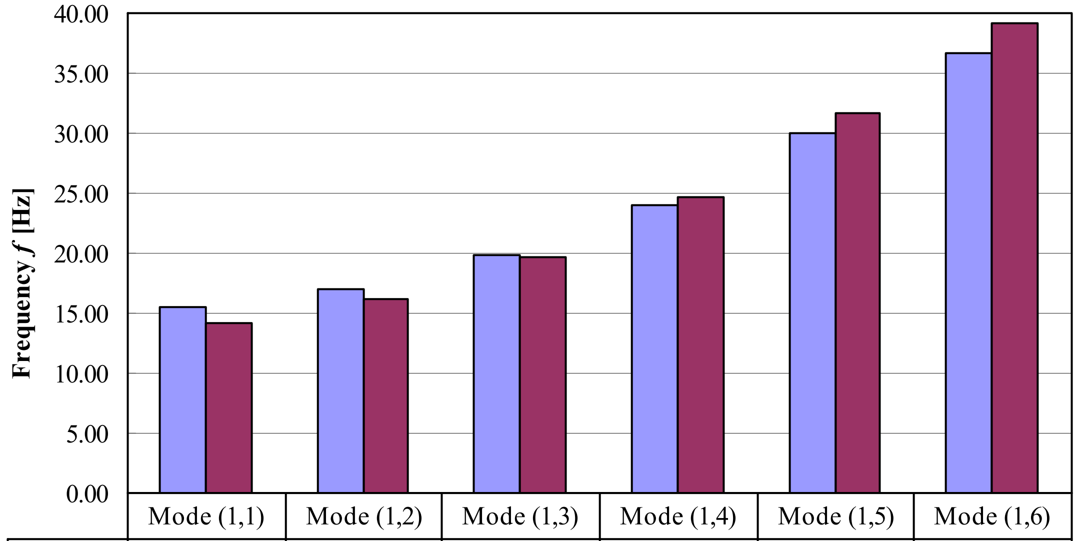
Figure 6.5 illustrates the comparisons of natural frequencies of the floors with and without ceiling (Floors E and F and Floors H and I). A reduction of the first and second natural frequencies, minor variation of the third ones and a reverse effect on the higher modes could be noted. The fundamental frequencies dropped by 7.57% and 7.89% while those of Mode (1,6) increased by 5.67% and 6.56% respectively. The spacing between neighbouring frequencies was raised. The variation in damping was again rather inconclusive. The shape of Mode (1,1) became less curved due to the addition of ceiling. As the addition of ceiling raised mass and stiffness of the system, both parameters impact the natural frequencies with varying degrees from lower to higher modes (see Section 6.4).



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor E	14.53	16.21	19.63	24.10	30.47	37.24
Floor F	13.43	15.52	19.78	25.12	32.17	39.35

(a) Floors with TR26 strongback of 35 mm × 97 mm

at mid-span and joist spacing of 600 mm



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor H	15.47	17.02	19.87	23.97	30.04	36.72
Floor I	14.25	16.16	19.74	24.69	31.69	39.13

(b) Floors with TR26 strongback of 35 mm × 97 mm

at mid-span and joist spacing of 400 mm

Figure 6.5: Natural frequencies of the floors with and without ceiling

The addition of ceiling also reduced the deflection under point load. This effect was more significant for the floor with 600 mm joist spacing where the deflection dropped from 1.25 mm for Floor E by 22.40% to 0.97 mm for Floor F. For the floor with 400 mm joist spacing the added ceiling reduced the deflection by 10.42% by comparing Floors H and I.

6.3 Effect of strongback

Figure 6.6 to Figure 6.9 show the comparisons of natural frequencies of the floors without and with strongback whereas the number, dimension or type of strongback varied. The impact of inserting strongback on the first two natural frequencies was rather negligible, independent of number, dimension or type of strongback used. For the other principal first order modes, the frequencies were raised with an increasing effect for successive modes. The spacing between adjacent natural frequencies became widened.

The addition of the strongback mainly decreased the damping of Modes (1,1) and (1,2) for the floors with 600 mm spacing (Table 6.2) and the damping of Mode (1,2) but not of Mode (1,1) for the floors with 400 mm spacing.

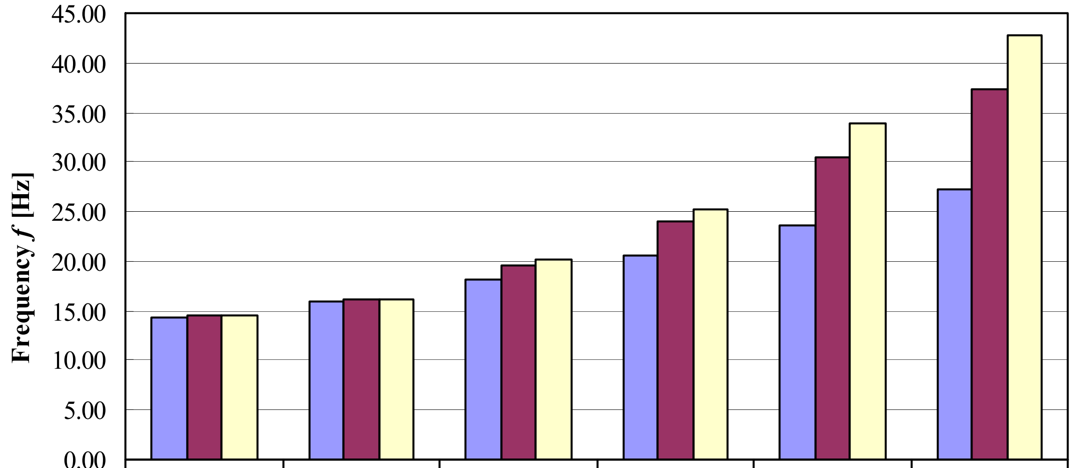
Using strongback resulted in a less curved shape for Mode (1,1) (Figure 6.1). In case of Mode (1,2), the shape also became less curved (e.g. Floor B) with clear anti-nodes at the edges. For Mode (1,3), the movement at the floor centre became less than that at the edges.

All floors showed notable reduction in deflection due to the use of strongback (Figure 6.2) as this raised the load sharing effect of joists. For the lower joist spacing, the addition of strongback at mid-span was slightly more effective as indicated by comparison of the deflection of Floors H and G with a reduction of 33.33%, and Floors E and A with a reduction of 30.56%.

More specific effects due to varied number, dimensions and type of strongback are detailed as follows. The observations are further discussed in Section 6.4.

6.3.1 Using different numbers of strongback

Figure 6.6 shows the natural frequencies of three floors with the same configuration except that Floor A had no strongback, Floor E had one strongback at mid-span and Floor D had a strongback at each third-span. Increasing the strongback number enhanced the effect that was found for inserting one strongback element. The frequency of Mode (1,5) could be raised from 23.71 Hz to 30.47 Hz when using one strongback at mid-span and to 33.81 Hz when installing strongback elements at third-spans. The number of first order modes below 40 Hz reduced from 8 for Floor A to 6 for Floor E and to 5 for Floor D.



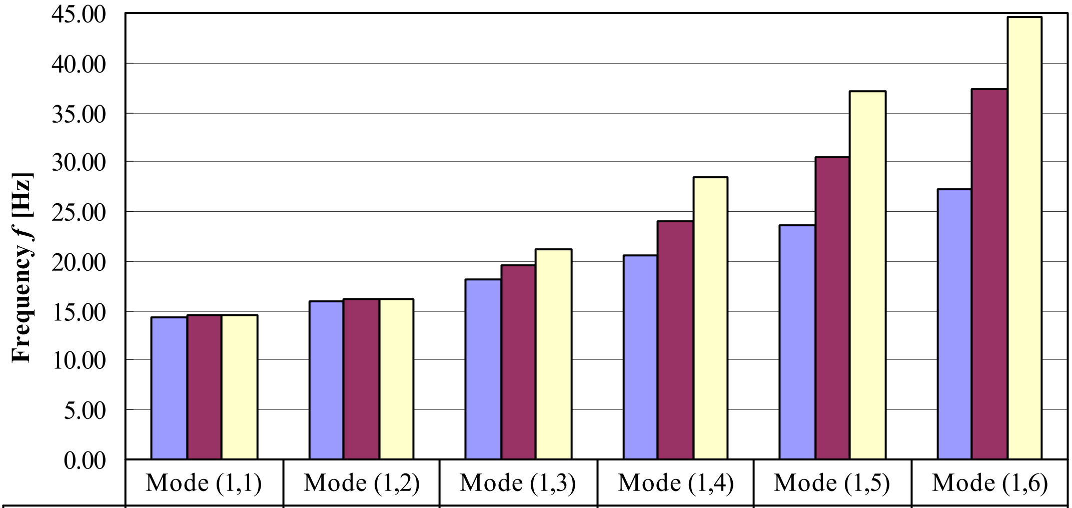
	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor A	14.42	16.01	18.07	20.60	23.71	27.28
Floor E	14.53	16.21	19.63	24.10	30.47	37.24
Floor D	14.49	16.18	20.09	25.27	33.81	42.82

Figure 6.6: Natural frequencies of floors with joist spacing of 600 mm without and with TR26 strongback of 35 mm × 97 mm at mid-span or third-spans

Using strongback at mid-span or third-spans of the floor clearly reduced the deflections at the floor centre. Floor E with one strongback at mid-span deflected 30.60% less than Floor A without strongback. The comparison of Floors D and A showed a similar reduction in deflection (28.89%) if installing strongback elements at third-spans.

6.3.2 Using different dimensions of strongback

Figure 6.7 illustrates the identified natural frequencies of Floor A without strongback, Floor E with strongback and Floor B with strongback of greater cross section. The strongback with larger cross section was highly effective, about double as much on the natural frequencies of Modes (1,3) to (1,6) as the one with smaller cross section. While e.g. the frequencies of Modes (1,3) and (1,5) for Floor E were 8.63% and 28.51% higher compared to Floor A, the ones for Floor B were 17.43% and 56.43% higher. The number of first order modes below 40 Hz could be reduced from 8 for Floor A to 6 for Floor E and to 5 for Floor B.



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)
Floor A	14.42	16.01	18.07	20.60	23.71	27.28
Floor E	14.53	16.21	19.63	24.10	30.47	37.24
Floor B	14.59	16.23	21.22	28.40	37.09	44.56

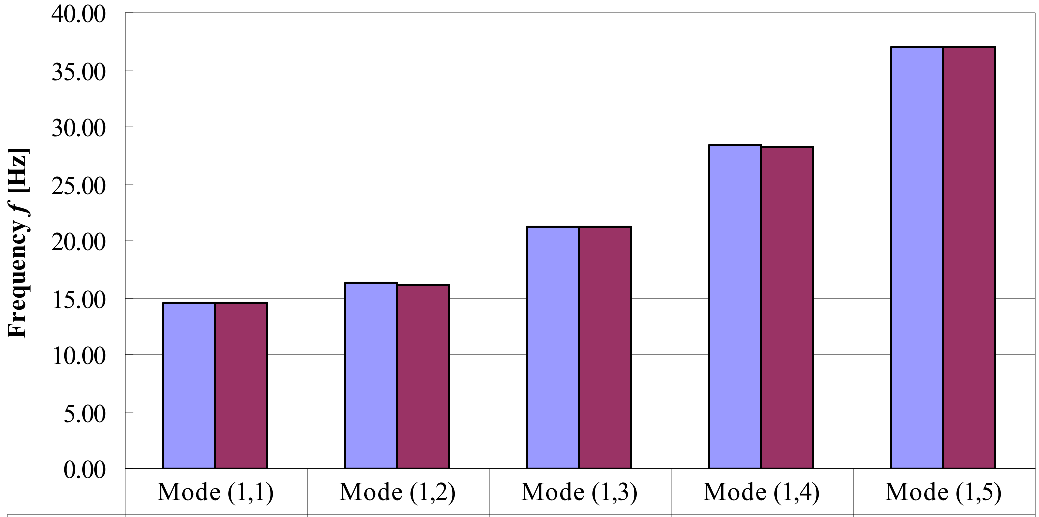
Figure 6.7: Natural frequencies of the floors without and with TR26 strongback of 35 mm × 97 mm and of 47 mm × 147 mm

From the comparisons in Figure 6.2, the installation of strongback at mid-span lowered the deflection by 30.56% for Floor E compared to Floor A. Replacing the strongback of 35 mm × 97 mm of Floor E with a greater strongback of 47 mm × 147 mm as for Floor B raised the efficiency in lowering the point load deflection with a reduction of 38.33%.

6.3.3 Using different material types of strongback

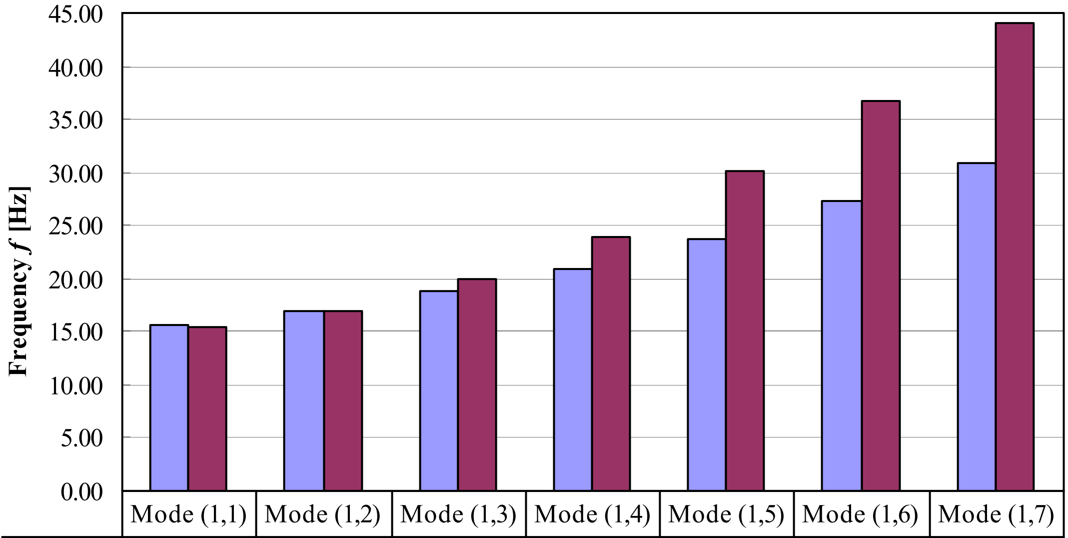
Floor B was constructed with TR26 solid timber strongback of 47 mm × 147 mm at mid-span. This strongback was then replaced by a Kerto S strongback of 45 mm × 147 mm (Floor C). Figure 6.8 shows that there was no significant difference in natural frequencies whether TR26 or Kerto S strongback of similar dimension was used. Also no significant differences in the damping ratios were generally found.

Reductions of 38.33% and 40.00% were found for the deflections, comparing Floor A to Floor B and Floor A to Floor C respectively.



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)
Floor B	14.59	16.23	21.22	28.40	37.09
Floor C	14.55	16.21	21.27	28.27	37.08

Figure 6.8: Natural frequencies of the floors with strongback TR26 of 47 mm × 147 mm or Kerto S of 45 mm × 147 mm



	Mode (1,1)	Mode (1,2)	Mode (1,3)	Mode (1,4)	Mode (1,5)	Mode (1,6)	Mode (1,7)
Floor G	15.54	16.99	18.80	20.97	23.77	27.37	30.92
Floor H	15.47	17.02	19.87	23.97	30.04	36.72	44.06

Figure 6.9: Natural frequencies of the floors with joist spacing of 400 mm without and with TR26 strongback of smaller dimension at mid-span

6.4 Discussion on the results of metal-web joist floor test series

The spacing between natural frequencies of Floor A was already below the critical value of 5 Hz for the principal first order modes below 40 Hz. Adding more joists to lower joist spacing increased the stiffness principally in longitudinal direction, which in turn raised the degree of anisotropy, and increased the mass. The rise in fundamental frequency showed that the effect of raised stiffness was dominating at the lower frequencies. Due to the higher degree of anisotropy, the spacing between adjacent natural frequencies mainly slightly decreased. The drop of the higher frequencies was however caused by the additional mass of the extra joists. This indicates that there was a stiffness effect on the first few modes which caused the natural frequencies to increase whereas the mass effect then cancelled the impact of increased stiffness and eventually became dominating at the higher modes. The reduction in the joist spacing increased the flooring stiffness so as to considerably reduce the deflection, in particular for the floors without ceiling. When ceiling was installed, the degree of reduced deflection due to narrower joist spacing was nearly halved.

The installation of ceiling added fairly the same amount of stiffness to the structure in both directions, which in turn resulted in a higher ratio of the stiffness in transverse direction to the one in longitudinal direction. Also the ceiling noggings further increased the transverse stiffness. This raised the spacing of neighbouring natural frequencies. The added mass from the plasterboards caused the lower natural frequencies to decrease whereas the effect of higher stiffness was dominating at the higher modes, causing the corresponding frequencies to increase. The raised flooring stiffness due to the installation of ceiling and noggins clearly reduced the deflection. The degree of this effect became halved for the floors with 400 mm joist spacing.

The effect of added ceiling on frequencies was thus contrary to the effect of reduced joist spacing. The trend for the deflection was fairly the same. However, also this modification may not necessarily improve the vibration performance of the floors.

The mode shape for Modes (1,1) became flatter and the movement at the floor centre for Modes (1,3) became lower than those at the edges by using ceiling or strongback. Although the movements at the edges may not actually be raised, they became more relatively significant for all of the first three first order modes compared to the floors without these structural elements.

Furthermore, using strongback would raise the degree of isotropy. The higher the number of strongback, the higher this degree would be. The separation between natural frequencies increased when installing the strongback elements. The effectiveness was stronger for the higher modes by even shifting some of the modes out of the critical frequency range. The spacing between the first two natural frequencies hardly varied. Fewer numbers of critical natural frequency separations were obtained when using a larger number of strongback elements or a bigger strongback cross section. The efficiency of the strongback of larger cross section at mid-span was generally more significant than the one of the smaller sized ones at third-spans.

There was no noteworthy difference in the effect on the dynamic performance whether TR26 or Kerto S strongback was used, neither on the vibration nor on the deflection. This may be due to not only the similar cross sections but also the similar moduli of elasticity.

For all floors with strongback compared to those without, the decrease in deflection was obviously caused by the increased stiffness in transverse direction. The change of the deflection at the floor centre was similar whether strongback elements were installed at mid-span or at third-spans of the floor. Using more strongback elements, which are placed properly separated, could be more efficient on the load sharing and on the reduction of deflections further away from the mid-span compared to using only one strongback at mid-span.

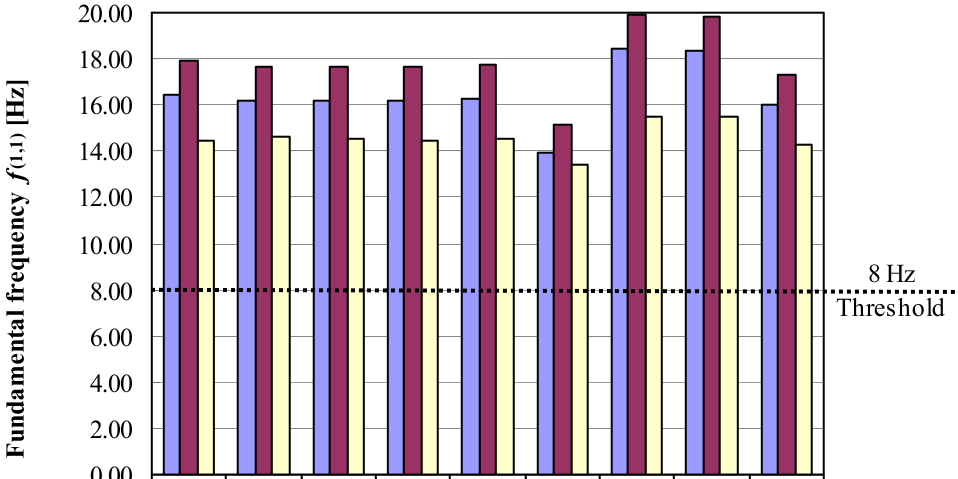
Therefore, using strongback enhances the vibrational performance of the flooring structure by further separating adjacent natural frequencies, reducing the number of critical modes, raising in particular the higher frequencies and lowering deflections. Doubling the amount of strongback material by using a single strongback element of larger cross section at mid-span or two elements of normal size at third-spans indeed amplifies these effects, with higher efficiency for the use of strongback of larger cross section at mid-span.

6.5 Comparison of measured results and predicted responses using EC5-1-1 criteria

The fundamental frequency of each MWJ floor was calculated without and with consideration of composite action of deck and joists. This composite action was also taken into account when calculating the deflection under unit point load and the unit impulse velocity response and its limit. The contributions of plasterboard and strongback to the transverse stiffness of the floor were considered as suggested in the UK NA to EC5-1-1 for determining the point load deflection.

6.5.1 Predictability of fundamental frequency of the MWJ floors

The measured and predicted frequencies are presented in Figure 6.10, together with the threshold value of 8 Hz. The measured fundamental frequencies were well above the threshold but all over-predicted. Without consideration of the composite effect the frequencies are over-estimated between 11.17% (for Floor B) and 18.66% (for Floor G) apart from Floor F by only 3.72%. If composite action is considered, over-estimations of 21.11% (for Floor B) to 27.99% (for Floor G) could be observed for all floors except Floor F with 12.96%.



Floor	A	B	C	D	E	F	G	H	I
Prediction (no comp.)	16.42	16.22	16.20	16.20	16.31	13.93	18.44	18.34	16.02
Prediction (comp)	17.88	17.67	17.65	17.65	17.76	15.17	19.89	19.78	17.28
Measurement	14.42	14.59	14.55	14.49	14.53	13.43	15.54	15.47	14.25

Figure 6.10: Comparison of predicted and measured fundamental frequencies

6.5.2 Predictability and suitability of parameters for determining the unit impulse velocity response

Three parameters, which are used to calculate the impulse velocity response and its limit can be determined from measurements: the number of first order modes below 40 Hz, the fundamental natural frequency, and the corresponding damping ratio.

The damping ratios corresponding to the fundamental vibration mode of all floors are shown in Figure 6.11, with the dashed line as the mean value. It can be seen that the variation is very small and all damping ratios are fairly close to the mean value. The largest discrepancies were found for Floors G and F, 10.47% lower and 15.12% higher, respectively. On average, all of these damping ratios are 14% below the (fixed) value of 1% suggested in EC5-1-1 and with 57% far below the (fixed) value of 2% suggested in the UK NA to EC5-1-1.

The number of first order modes below 40 Hz is under-estimated in seven of nine cases as shown in Figure 6.12. The difference in the mode number is under one for most floors.

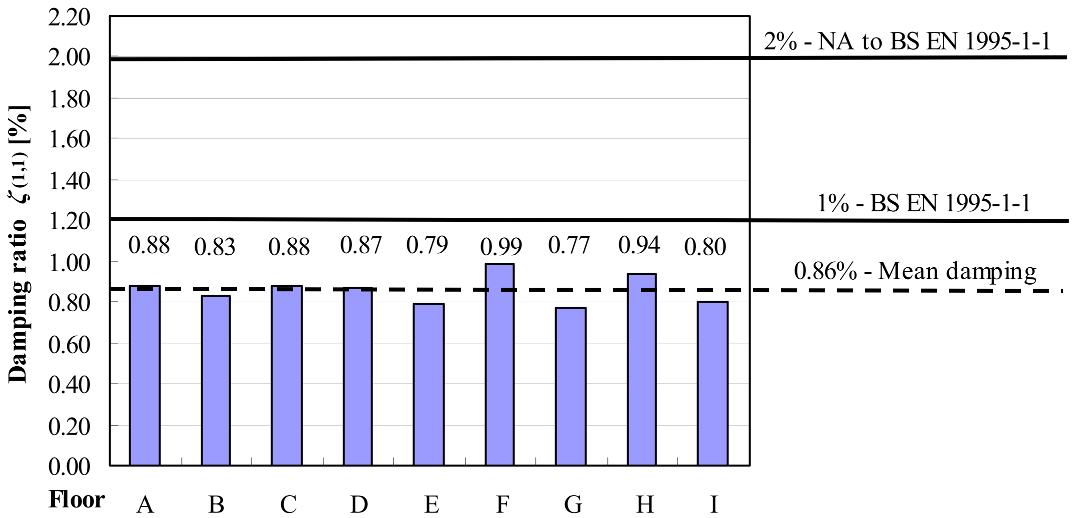
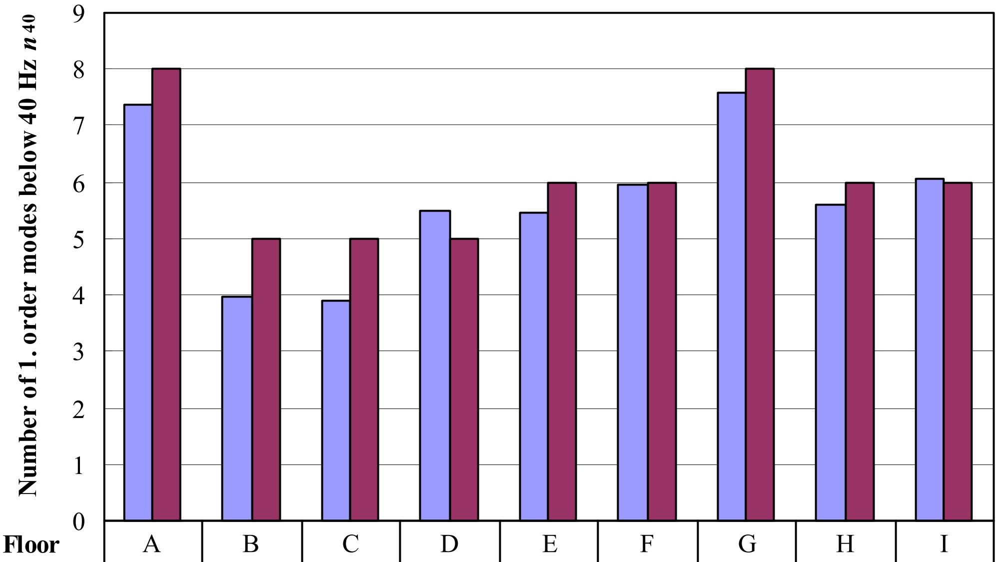


Figure 6.11: Damping ratios of the fundamental vibration modes, together with the mean and suggested design values from EC5-1-1 and the UK NA to EC5-1-1

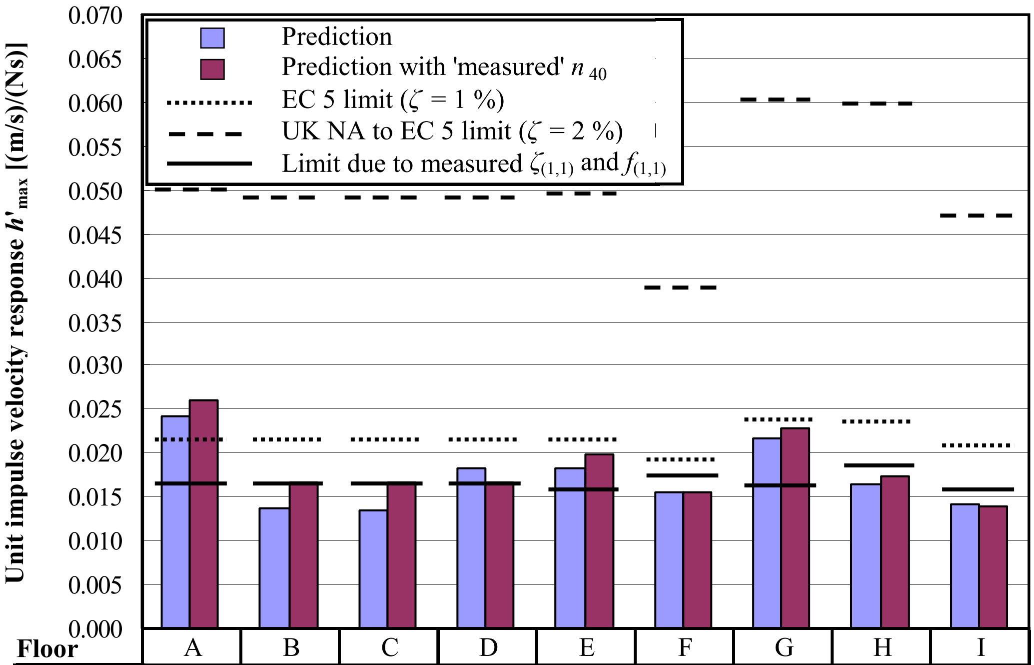


Floor	A	B	C	D	E	F	G	H	I
Prediction	7.4	4.0	3.9	5.5	5.5	5.9	7.6	5.6	6.1
Measurement	8	5	5	5	6	6	8	6	6

Figure 6.12: Comparison of predicted and measured number of first order modes below 40 Hz

The velocity responses were determined twice, using initially the calculated numbers of first order modes below 40 Hz and then secondly the numbers of those modes identified from the measured results (Figure 6.13). There are three different limiting values for each floor included in the figure. The dotted line reflects the limit according to EC5-1-1 which suggests a damping ratio of 1% to be used (unless other values being proven to be appropriate). The UK NA to EC5-1-1 suggests a damping ratio of 2% instead. The corresponding limit is illustrated by the dashed line. The continuous line represents the limit which was calculated using the same design rule but with the measured fundamental frequency and the corresponding damping ratio.

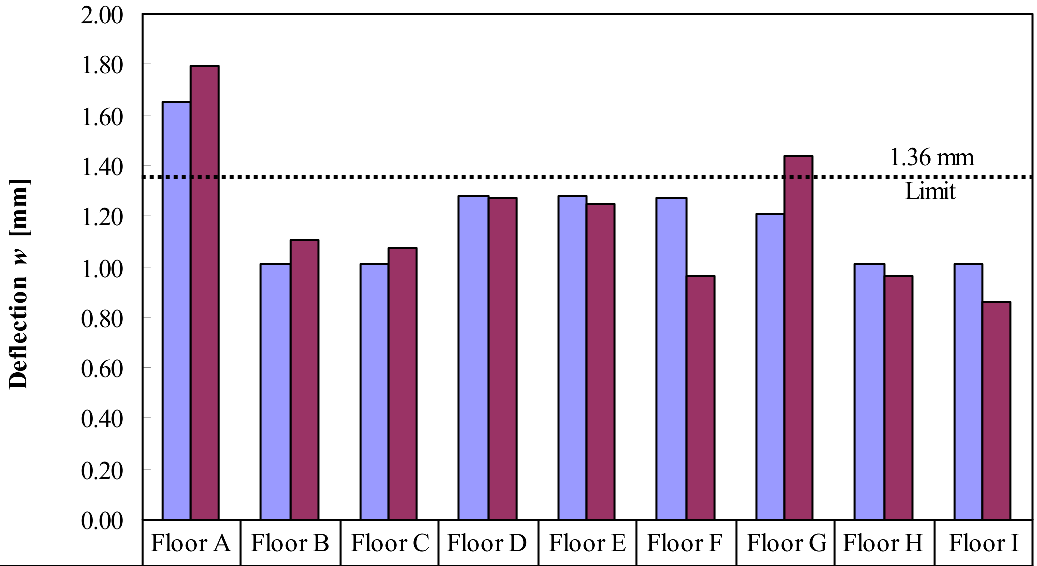
All the calculated velocities are far below the limit from the UK NA to EC5-1-1. Using the damping ratio suggested in EC5-1-1 causes Floor A to fail and the other floors to be safe, some marginally with the velocities slightly below the limit. The most realistic option in which parameters determined from measurements are used to calculate the velocity limit led to four out of nine floors (Floors A, B, E and G) to be beyond the acceptability (Figure 6.13).



Floor	A	B	C	D	E	F	G	H	I
■	0.02417	0.01373	0.01341	0.01813	0.01824	0.01539	0.02172	0.01640	0.01413
■	0.02605	0.01673	0.01670	0.01670	0.01984	0.01551	0.02282	0.01740	0.01401
....	0.02181	0.02159	0.02157	0.02157	0.02169	0.01922	0.02395	0.02383	0.02121
– – –	0.05017	0.04918	0.04908	0.04908	0.04961	0.03897	0.06050	0.05989	0.04745
——	0.01712	0.01667	0.01721	0.01706	0.01618	0.01761	0.01655	0.01867	0.01612

Figure 6.13: Velocity responses and limits determined with and without measured parameters

Figure 6.14 shows the predicted and measured deflections and their limits. The deflections for four floors were over-estimated by up to 30.92% (Floor F), for four other floors under-estimated by up to 15.97% (Floor G), and closely predicted for one floor. One of the nine floors (Floor G) was misclassified as acceptable based on the design values, but regarded as unacceptable based on the measured parameters.



	Floor A	Floor B	Floor C	Floor D	Floor E	Floor F	Floor G	Floor H	Floor I
Prediction	1.65	1.02	1.02	1.28	1.28	1.27	1.21	1.02	1.02
Measurement	1.80	1.11	1.08	1.28	1.25	0.97	1.44	0.96	0.86

Figure 6.14: Comparison of predicted and measured point load deflections at the floor centre

6.5.3 Discussion of predictability of vibration parameters of MWJ floors

The deflection of flooring systems cannot always be predicted accurately, which can result in misclassifying flooring systems as acceptable while the vibration performance is not satisfactory. Here concentration was focused on the deflection at the floor centre. The maximum mid-span deflection at the loading point of floors not supported along all sides can be expected at the unsupported edges, as confirmed by the test results (see Table 6.3 and Table 6.4). Comparing the corresponding measured deflection to the design limit would classify the metal-web joist floors as unacceptable. This would be the case for exactly following the design rules in EC5-1-1 as it refers to the maximum deflection at any point on the floor structure by assuming that the EC5-1-1 refers to the deflection on the joists.

As long as the fundamental frequency is well above the threshold level of 8 Hz, the over-estimation is found not to be crucial if a separate consideration of this criterion is undertaken. It becomes more crucial in conjunction with the impulse velocity response requirement.

The velocity limit will increase if frequency or damping is increasing, see the design equation (7.4) in EC5-1-1. This shows that especially the damping ratio suggested in the UK NA to EC5-1-1 is leading to easy fulfilment of the velocity criterion. Also the constant over-estimation of the fundamental frequency will raise the velocity response limit. The relative variations of frequency and damping have the same crucial degree of influence on the velocity design criterion. The damping ratio recommended in the UK NA to EC5-1-1, however, is 100% higher than the one suggested in EC5-1-1, which is still above the damping measured. The dependence on three factors, which include two unsafely determinable ones (permanently over-estimated frequencies and usually underestimated numbers of first order modes) and one pre-defined parameter (over-estimated damping), makes the velocity criterion not a reliable design rule.

As the EC5-1-1 states that a damping ratio of 1% should be used unless other values are proven to be appropriate, it is suggested that the damping determined from the measurements should be adopted for metal-web joist floors (see Chapter 8).

7. Investigation of British and Finnish flooring systems with respect to EC5-1-1 design rules

A Finnish flooring structure and a British flooring structure were examined with respect to the design rules of EC5-1-1 only, EC5-1-1 with the UK NA to EC5-1-1 and with the Finnish National Annex to EN 1995-1-1:2004 (FI NA to EC5-1-1) (VTT 2007). Comparisons were made for the calculated and measured parameters including the limits and for the design to the nationally determined parameters (NDPs) and criteria. This was part of the aforementioned STSM. The two floors were tested before the start of the STSM.

7.1 Investigation of the Finnish flooring structure

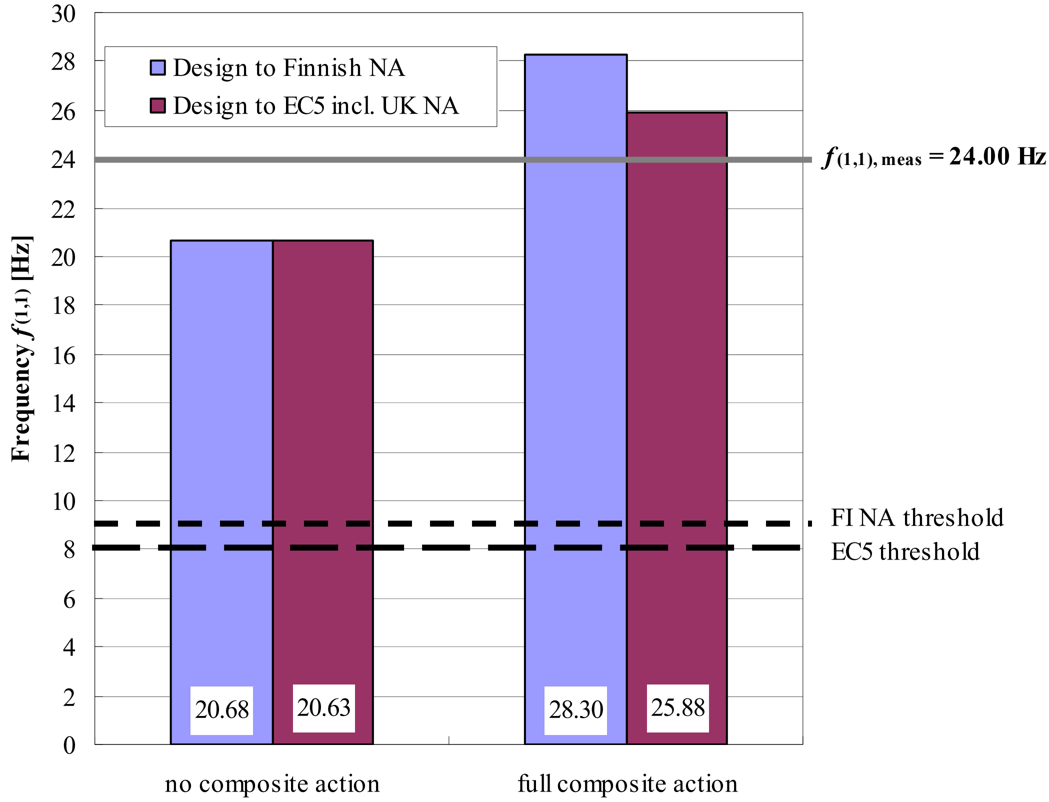
The Finnish test floor had dimensions of 6.0 m × 4.3 m and was constructed with LVL joists of 51 mm × 400 mm, spaced at 600 mm. The ends were connected to LVL rim boards, which had the same dimensions as the joists. The floor was simply supported on timber walls along all four sides and decked with 18 mm thick plywood boards, which were connected to the joists using glue and screws. A 60 mm concrete screed was added on top of a 30 mm thick hard mineral wool (ASL2) layer that was placed on the plywood deck. The concrete screed had no structural connection to the flooring structure. LVL blockings, staggered between the main LVL floor beams at the third span points and glued to the deck, and tension bars below the rows of blockings were used as transverse stiffeners. The plywood decking layer and the tension bars were the only continuous primary floor structural elements in the transverse direction. However, also the concrete screed helped to distribute the load in the direction perpendicular to the span.

The Finnish flooring structure was assessed at two different construction stages: before and after the concrete screed was added to the flooring system. Due to the plywood layer being glued to the floor joists, full composite action of plywood deck and joists was assumed. Since the EC5-1-1 does not give any advice for consideration of composite action, and this effect may sometimes be neglected in calculations for simplicity, the calculations for the fundamental natural frequency were repeated where composite action was not considered. For all other calculations composite action was included. To determine the stiffness due to blocking and tension bars in the transverse direction, composite and non-composite I-sections were assumed to be effective, where the tension bar acted as the bottom flange, the plywood deck as the top flange and the blocking as the web. For not overestimating the transverse stiffness, however, a zero width was assumed for the web.

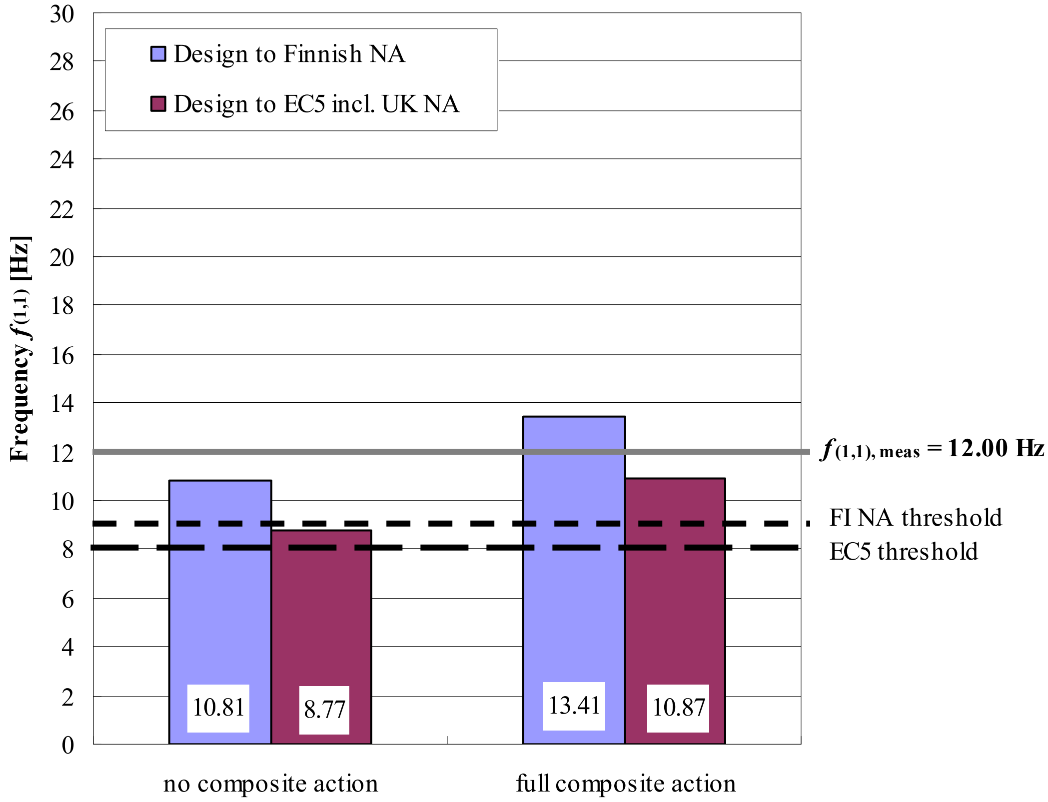
Figure 7.1 shows that the predicted frequencies were significantly higher when full composite effect was considered. Furthermore, the concrete screed drastically reduced the fundamental frequencies in a range that becomes critical for design. The predicted frequencies based on the design rule of the FI NA to EC5-1-1 for floors supported along four edges and of the simplified design rule of EC5-1-1 were very close if only the timber structure was considered but composite action neglected. For the floor without concrete screed but with consideration of composite action and for the floor with concrete screed, the calculations to the Finnish formula resulted in higher fundamental frequencies by up to 23% compared to the calculations to the EC5-1-1 formula. For the flooring structure without concrete screed, the fundamental frequency was well above the thresholds and over-predicted if full composite effect between the deck and joists was assumed (Figure 7.1(a)). For the flooring structure with concrete screed, the predicted frequency under consideration of composite action and with respect to Finnish design was the only one over-predicting the measured value (Figure 7.1(b)).

From Figure 7.2, the predicted deflections were 52% and 217% larger than the measured ones with respect to UK design. Using the Finnish criteria could closely predict the actual deflection in absolute terms. Although the added concrete screed had halved the actual floor deflection in comparison with the floor without the concrete screed, the predicted deflection remained unchanged among the two different structures with respect to the equation of the UK NA to EC5-1-1. The deflections calculated using the UK design rule were 73% and 138% higher than those determined from the Finnish formula. In all cases, the floors would have satisfactory performances although the limits showed that the UK design criterion is much more generous since the allowable deflection was more than double as much compared to the limit regarding Finnish design.

The velocity criterion is not included in the Finnish design guide but is given in EC5-1-1 and the UK NA to EC5-1-1. Figure 7.3 shows that the allowance given by the UK NA to EC5-1-1 was much higher than the one given by the EC5-1-1. The addition of the concrete screed considerably reduced the velocity response and the limits. However, the velocity response was not a crucial criterion in this example.

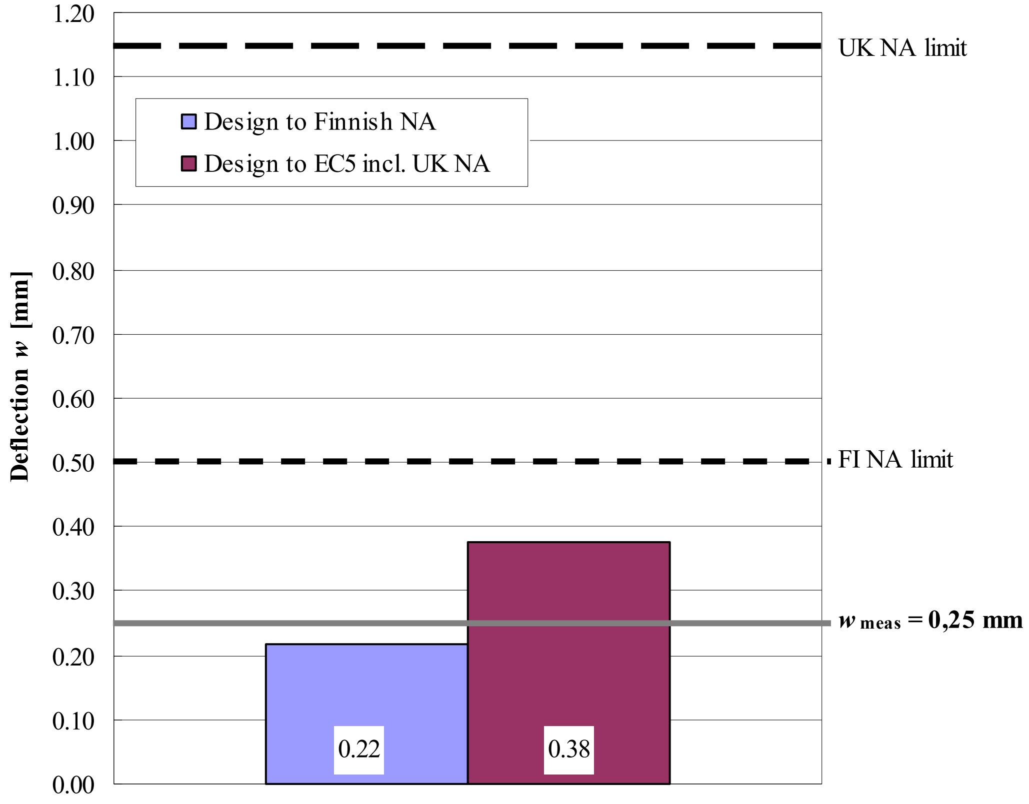


(a) Without concrete screed

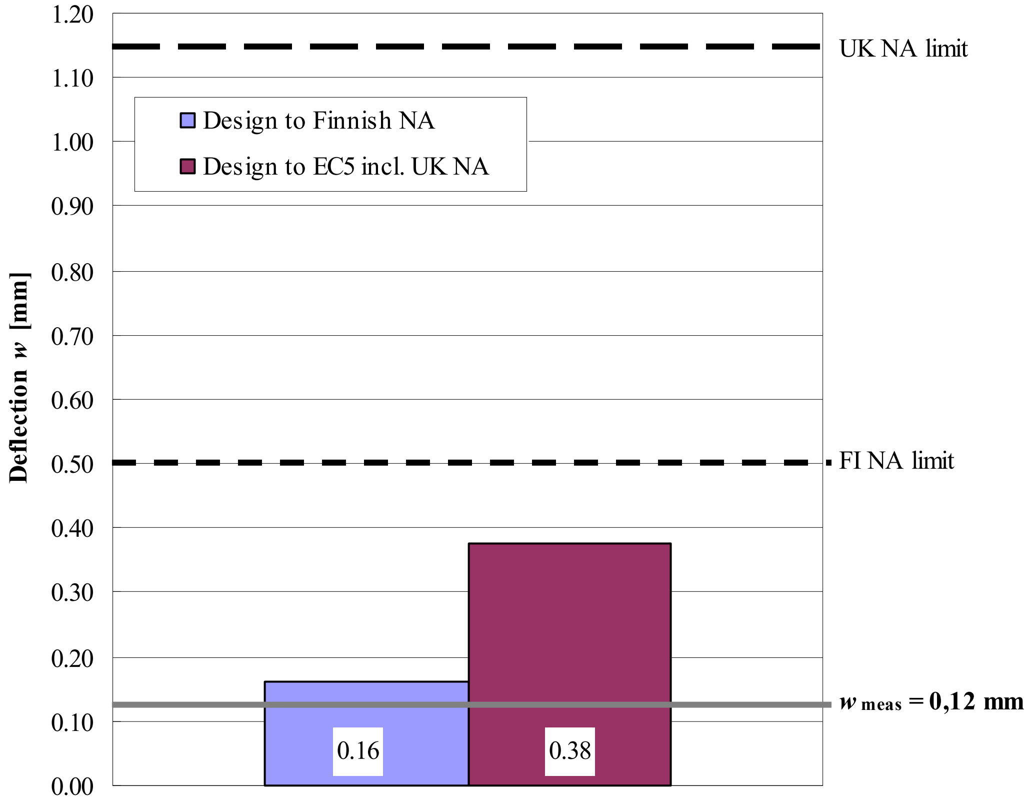


(b) With concrete screed

Figure 7.1: Predicted and measured fundamental frequencies with respect to the design thresholds in Finland and UK for the Finnish floor



(a) Without concrete screed



(b) With concrete screed

Figure 7.2: Predicted and measured point load deflections with respect to the design limits in Finland and the UK for the Finnish floor

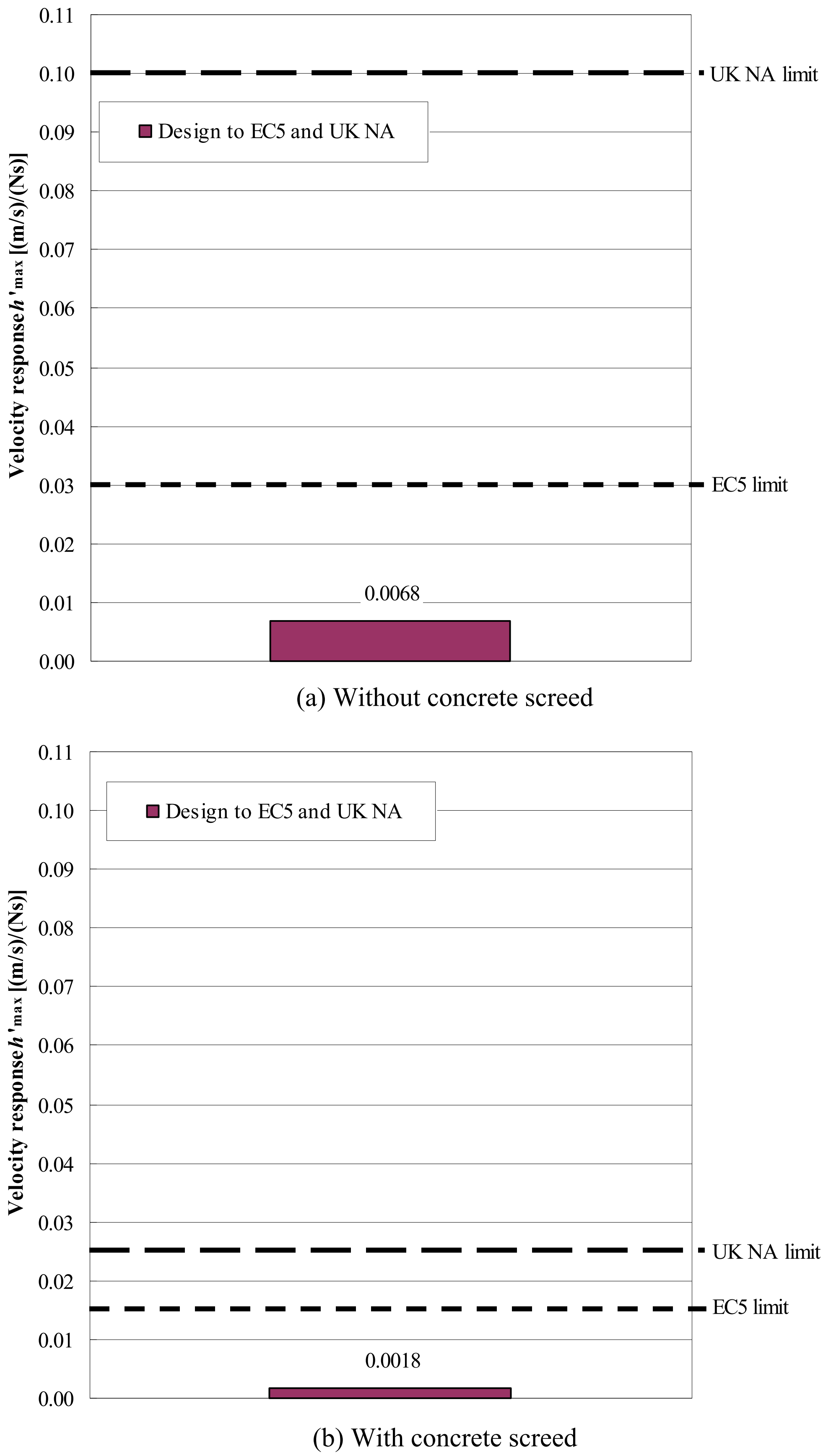


Figure 7.3: Predicted velocity responses and limits with respect to EC5-1-1 and the UK NA to EC5-1-1 for the Finnish floor

7.2 Investigation of the British flooring structure

The British flooring system selected for this investigation was Floor JJI 2 O. The deck of the British flooring structure was fixed to the joists using screws, which means that a certain degree of composite action was achieved. The fundamental frequencies were calculated by assuming no composite action first and then an appropriate level of composite action due to the screw-fixing. In all other calculations composite action was accounted for. The degree of composite action was calculated using the guidelines in EC5-1-1 (Section B.2 in EC5-1-1). The formulae for calculating the natural frequencies of the British floor were the same for Finland and the UK since this floor was supported along two edges only so that there are no differences in frequency predictions (see Table 3.6).

All calculated natural frequencies lay well above the threshold levels (Figure 7.4), but it should be mentioned that the flooring structural elements were slightly oversized. However, no matter whether composite action was considered, the measured fundamental natural frequency was over-predicted, by up to 22%. The estimation of the fundamental frequency influences also the estimation of the impulse velocity response limit. According implications are discussed in previous Section 6.5.3 and Chapter 9.

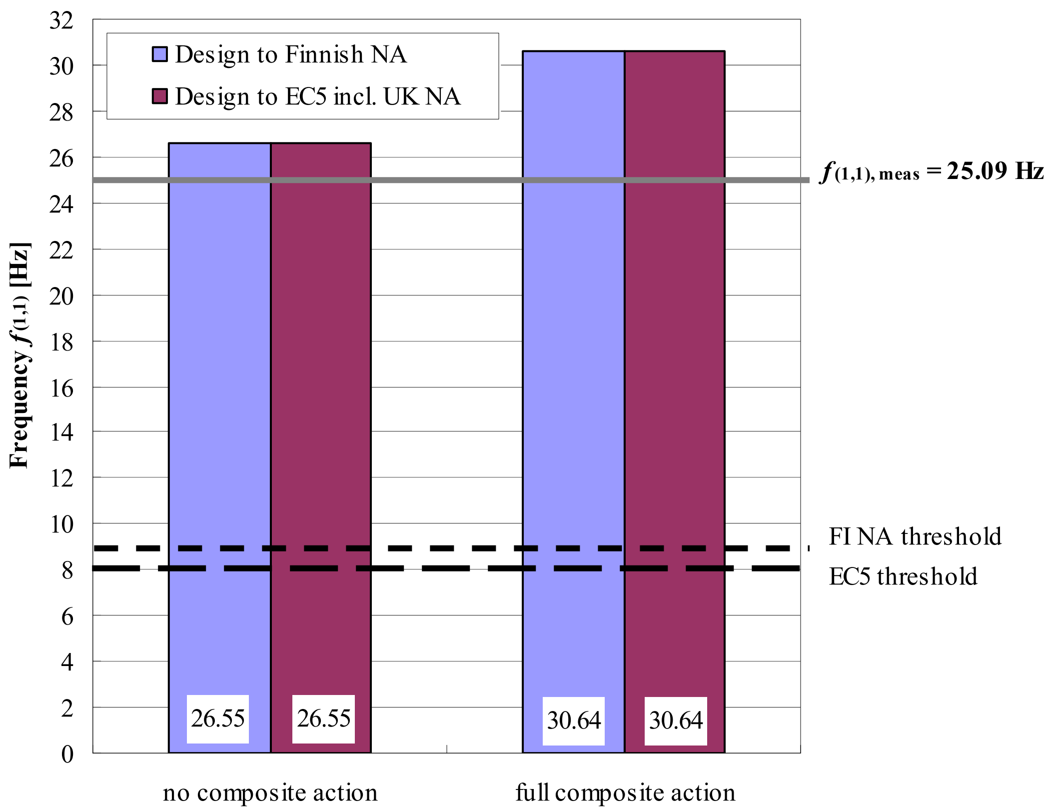


Figure 7.4: Predicted and measured fundamental frequencies with respect to the design thresholds in Finland and the UK for the British timber floor

Figure 7.5 shows that the design to the FI NA to EC5-1-1 provided relatively accurate prediction of the unit point load deflection, and the design to the UK NA to EC5-1-1 underestimated the deflection. Whereas the predicted and measured deflections were well below the UK limit and would thus be regarded to be acceptable in the UK, they were well above the Finnish limit and would thus be regarded to be unacceptable in Finland.

Figure 7.6 shows that the velocity response was not critical for design in this example. However, the corresponding limit to the UK NA to EC5-1-1 was far more generous than the one to EC5-1-1.

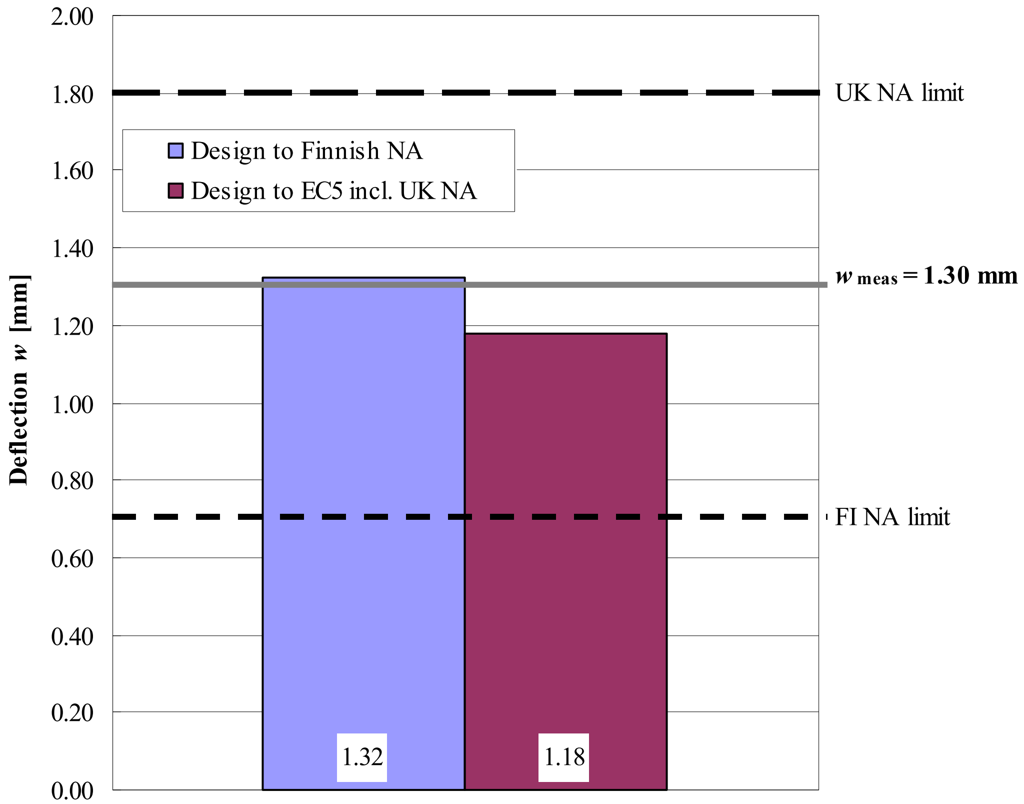


Figure 7.5: Predicted and measured point load deflections with respect to the design limits in Finland and the UK for the British timber floor

7.3 Summary of investigations of Finnish and UK floor design

For the classification of a flooring system as high-frequency floor in Finland, the fundamental natural frequency needs to be above a threshold of 9 Hz that is more than 12% above the EC5-1-1 (and thus UK) requirement. The design then relies on a static deflection criterion where the allowable deflection is usually well below the UK limit. In the EC5-1-1 and the UK NA to EC5-1-1, a velocity response criterion is included additionally.

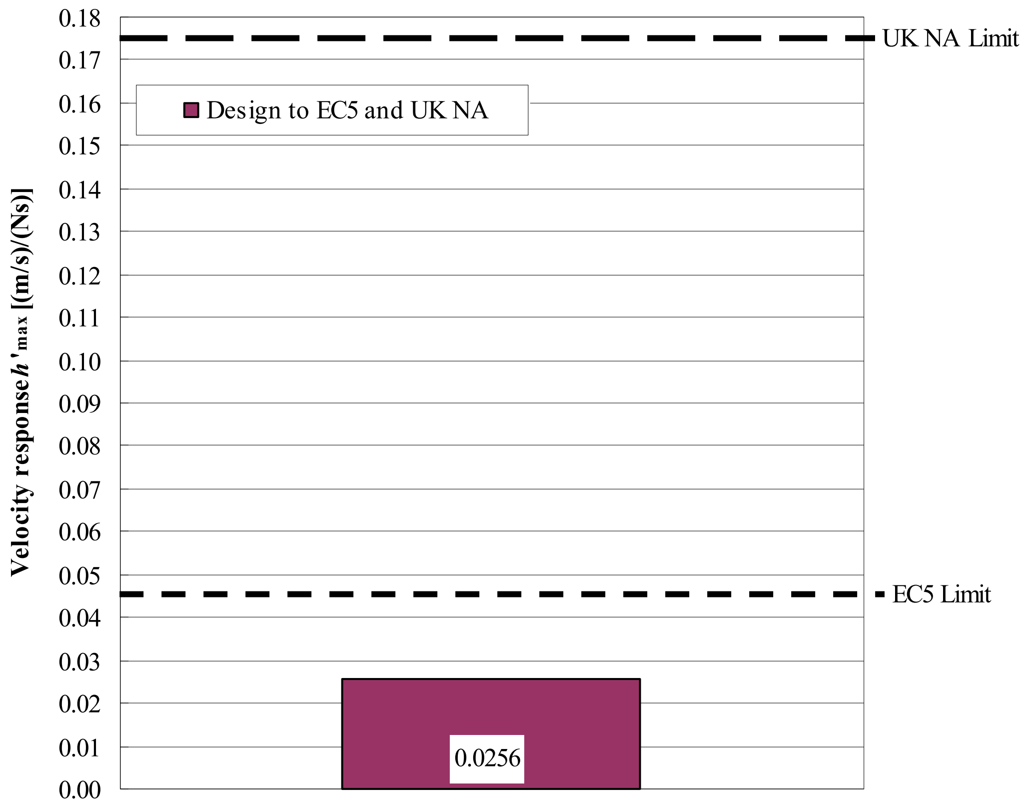


Figure 7.6: Predicted velocity responses and limits with respect to EC5-1-1 and the UK NA to EC5-1-1 for the British timber floor

The simplified formula in EC5-1-1 for calculating the fundamental natural frequency for floors supported at four edges gave similar predictions as the precise formula of the FI NA to EC5-1-1 if neglecting the composite action of the timber structure. Otherwise, using the Finnish formula led to predictions, which could be considerably higher than those using the simplified formula. The measured unit point load deflections of the two investigated Finnish flooring structures were overestimated by at least 73% based on the UK design equation. In one case, the prediction using the Finnish formula was slightly below the measured value, and in the other case slightly above. Both criteria yielded predictions close to the measured results for the UK timber floor, but the deflection was under-predicted to the UK NA to EC5-1-1 and may thus not be on the safe side. The velocity limit to the UK NA to EC5-1-1 is more generous than that to the EC5-1-1 and was 65% to 289% higher in the examples shown.

The conducted calculations indicated that it is a complex task to make accurate assumptions for determining the transverse stiffness when, beside the decking layers, transverse stiffening elements are used. Simple guidance for estimating the crosswise stiffness is neither provided in the Eurocode nor properly defined in the National Annexes of Finland and the UK. It was also shown that it is not clear whether composite action should be accounted for in the calculation of the fundamental natural frequency.

7.4 Concluding remarks on the study of vibrational floor design in the UK and Finland

The research undertaken during the STSM showed the differences in the design, construction and assessment methods in the UK and Finland, of which the major part was presented in this section. In general the Finnish design rules on vibration are stricter than those of the UK, the latter being even more generous than the criteria in EC5-1-1. Whereas all three flooring types investigated showed satisfactory performances with regard to the UK design rules, one of the systems was classified as unacceptable to the Finnish standards. Nevertheless, neither in UK nor in Finland can the dynamic parameters be predicted accurately in all cases, which could result in misclassification of flooring structures.

Reconsideration of the design rules and guidance for accurately determining the transverse stiffness and recommendations regarding consideration of composite effects are needed. Future research is also needed to show whether different construction practices justify different design methods in different countries. The procedures for more accurately predicting the floor performances, thresholds and limits need to be further harmonised.

Furthermore, parametric studies on the velocity response criterion currently used in the UK are required to assess whether this design rule is redundant or should be modified due to a relatively high limiting value, which may satisfy this criterion easily in common cases. This may then need further reconsideration of the given set of design criteria regarding dynamic floor performances in the EC5-1-1 to produce appropriate guidance for the UK NA to EC5-1-1.

8. Statistical analysis of damping

Damping plays an important role in floor vibrations as it influences the vibration amplitude of structures and is used as a design parameter in standards and guides (e.g. EC5-1-1; NA to EC5-1-1; Ohlsson 1988). The amplitude damps out more quickly and occupants will become less sensitive to initial vibration velocities if the damping is increasing (Lenzen 1966, Ohlsson 1988). However, as was shown in the literature review (Chapter 3), the determination of the damping is complex, depending on test procedures and data analyses. The damping ratios suggested for use in serviceability design of timber flooring structures vary considerably. The choice of a damping ratio can largely influence the judgement of classifying flooring structures as satisfactory or unsatisfactory regarding their dynamic performances.

There is an uncertainty about the damping ratios chosen for design, e.g. 1% given in the EC5-1-1 and 2% recommended in the UK NA to EC5-1-1. These damping ratios are used in design of all timber flooring structures. However, damping ratios may vary from structure to structure and also from vibration mode to vibration mode, so this needs to be considered. The vibration sensed by occupants is likely a collective effect of responses from different modes and in addition a question of the sensing person's location on the floor.

In this section it is attempted to more differentiate between damping ratios for certain structural conditions and for varying modes. Advanced analysis technique was used to identify damping ratios from measurements so as to estimate damping with a higher degree of accuracy (Section 4.5). Validation of this method can be found in e.g. Peeters and De Roeck (2001). The chosen number and locations of measurement points on the floor surface provided a good representation of the floor responses to reliably quantify the global property of modal damping. Each damping ratio analysed for each floor was the mean value of the measured responses corresponding to the selected projection channels.

8.1 Grouping damping ratios with dependence on certain structural conditions

Even though much care was taken during the testing and analysis process, it would be still hard to determine the damping of a structure with an optimum degree of accuracy. Dependent on certain conditions, therefore, the damping ratios of individual modes were grouped and presented in histograms together with density functions for normal distributions for the floors without imposed load (Figure 8.1 to Figure 8.3). The mean damping ratio, standard deviation and 5 percentile accompany the figures (Table 8.1 to Table 8.3). The mean value is the average of all damping ratios for each mode of one group. The standard deviation specifies the dispersion of the damping ratios around the mean value. The 5 percentile is the statistical value below which 5% of all damping ratios fall.

Although the influence of structural modifications on damping was not always conclusive, some dependence of damping on structural and non-structural details was identified. The damping ratios were accordingly categorised based on two main characteristics, each with two sub-divisions:

- **Joist-type**
    - Timber I-joists
    - Metal-web joists (with timber flanges)
- **Support condition**
    - 2-side supported
    - 4-side supported (all sides supported)

Thus, there will be four different groups, each matching one criterion of each category. From the previous sections it was obvious that some structural modifications were influential on damping, whose effect is not directly covered by these two main categories, e.g. floor length and joist depth, or the use of glue. It would become complex to define contribution of each structural detail with respect to damping. The basic approach was hence to section the damping ratios of the floors into the defined categories, which revealed most significant differences in damping for their sub-groups. The non-structural modification of imposed load was also highly influential on damping and was considered separately (Figure 8.4 and Table 8.4 to Table 8.6).

For the histograms, the damping ratios were grouped with increments of 0.2% for the floors constructed with metal-web joists and 0.5% for those constructed with I-joists. The wider steps for the damping of I-joist floors were selected due to a higher diversification of the damping ratios compared to those of the metal-web joist floors. The different selected steps only had an effect on the representation of the number of damping ratios in the histograms but did not affect the statistical properties.

8.2 Results of the statistical analysis

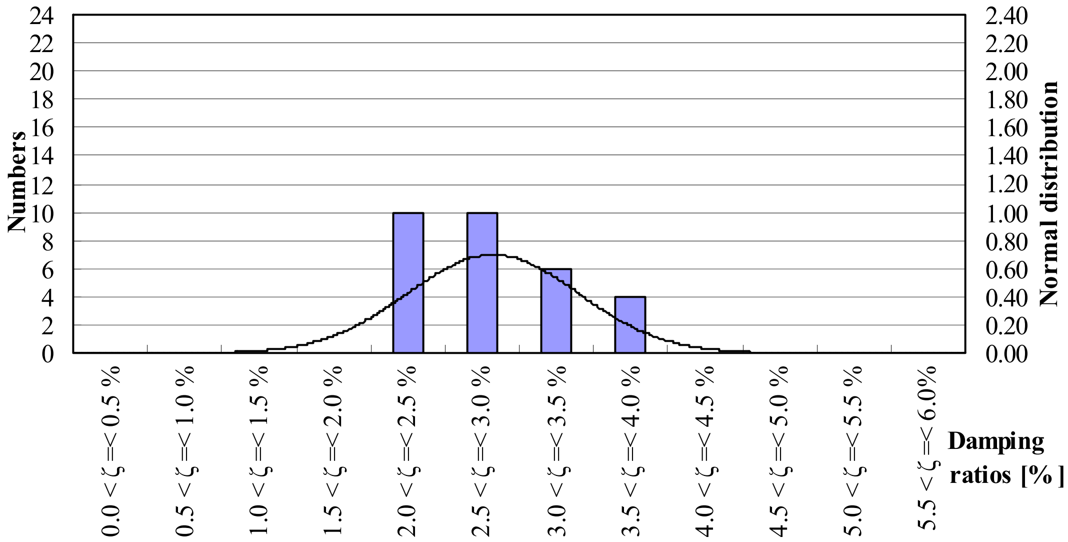
Table 8.1 presents the minimum and maximum damping ratios, the mean damping ratio, the standard deviation and the 5 percentile value for Modes (1,1) to (1,5) of all thirty tested two-side supported I-joist floors without imposed mass. The histograms and density functions of the damping ratios of the individual modes are presented in Figure 8.1. Those for all eighteen tested four-side supported I-joist floors are presented in Table 8.2 and Figure 8.2 and for all tested nine metal-web joist floors in Table 8.3 and Figure 8.3. It should be mentioned again that sometimes not all damping ratios were identified for all modes, which may reduce the total number of damping ratios regarding a particular mode. Certainly, the number of damping ratios for a specific group or mode is sometimes rather low, but the statistical analysis can still be expanded in future and thus optimisation can be obtained.

Table 8.1 indicates that the mean and 5 percentile values of the unloaded two-side supported I-joist floors were similar for Modes (1,2) to (1,5), but the values corresponding to Mode (1,1) were more than twice as much. Also the standard deviation was higher for the latter, which means that for two-side supported timber I-joist floors the range of damping ratios corresponding to Mode (1,1) would be more widely spread. For this mode the damping ratios, which were all above 2%, had a 5 percentile value slightly above 2%. All the higher modes, whose damping ratios were all below 2% (with one exception), had 5 percentile values of about 0.9%.

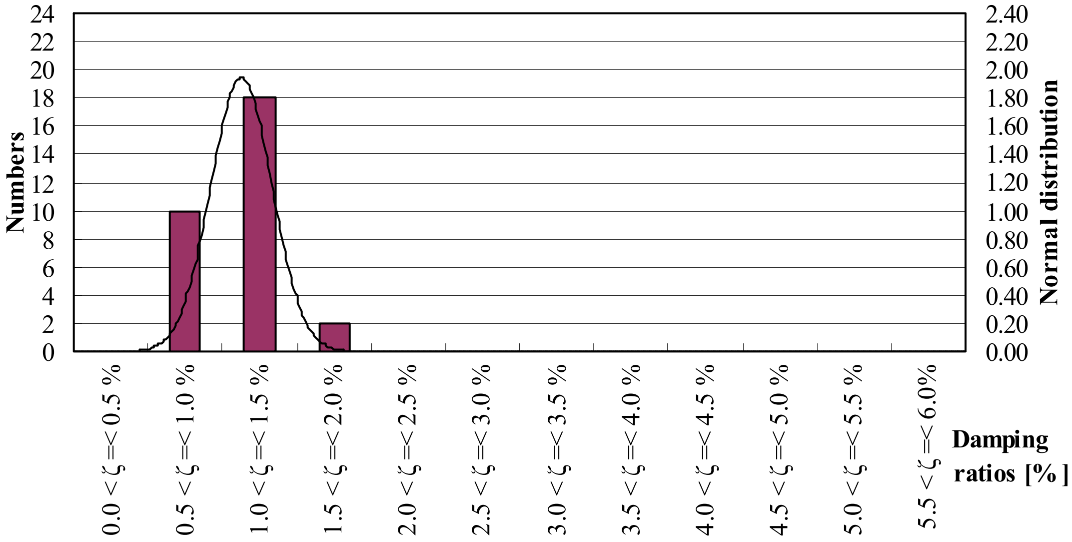
The mean damping ratios of the unloaded four-side supported I-joist floors were all similar with a minimum of 1.52% whereas the highest mean value of 1.89% again corresponded to the fundamental vibration mode, in this case Mode (1,2) (Table 8.2). The 5 percentile values varied between 1.07% and 1.49% among the modes. The standard deviations of the damping ratios corresponding to Modes (1,2) to (1,5) of two- and four-side supported floors were found to be similar. Since Mode (1,1) was not found for the four-side supported floors, it can only be assumed that an increase in damping as observed for Mode (1,2) could also be expected for Mode (1,1), but the degree of the effect may be lower. While for Mode (1,1) the movement was lowest at unsupported floor edges, Mode (1,2) usually possessed anti-nodes at those locations. The low movement at free edges for Mode (1,1) may result in lower friction compared to Mode (1,2) when introducing the additional supports under the outer floor joists. Hence, the increase in damping for Mode (1,1) may not had been as strong as for Mode (1,2). On the other hand, Mode (1,1) was not found for the floors supported along all four edges, which might be the result of an extraordinary high damping effect on this mode.

Table 8.1: Statistical analysis of damping ratios of unloaded two-side supported I-joist floors

ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	2.01	0.85	0.80	0.87	0.86
Maximum	3.98	1.70	1.98	1.93	2.38
Mean	**2.82**	**1.13**	**1.23**	**1.16**	**1.25**
Std Deviation	0.57	0.21	0.33	0.23	0.36
5 percentile	**2.03**	**0.90**	**0.93**	**0.92**	**0.92**

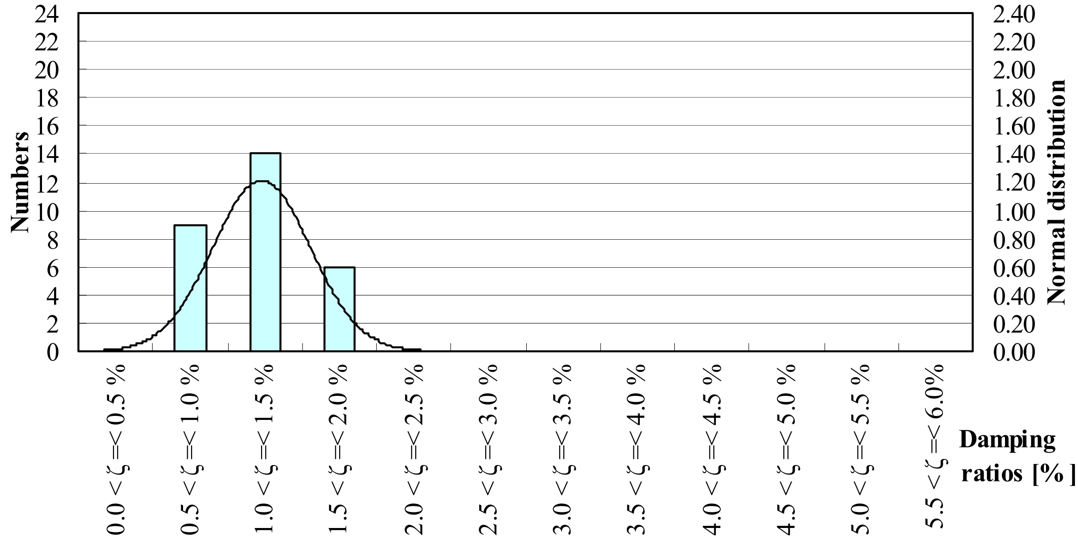


(a) Mode (1,1)

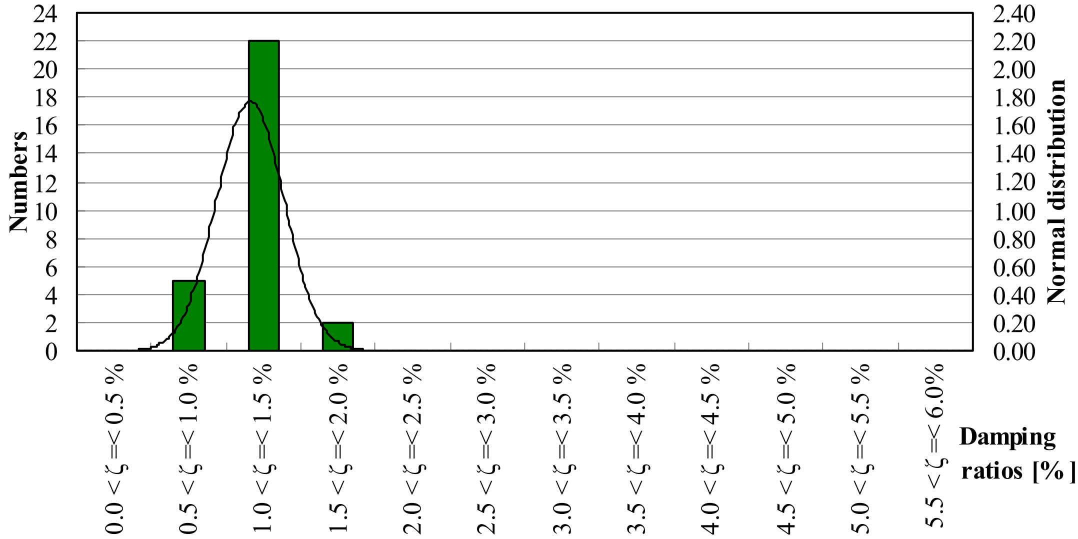


(b) Mode (1,2)

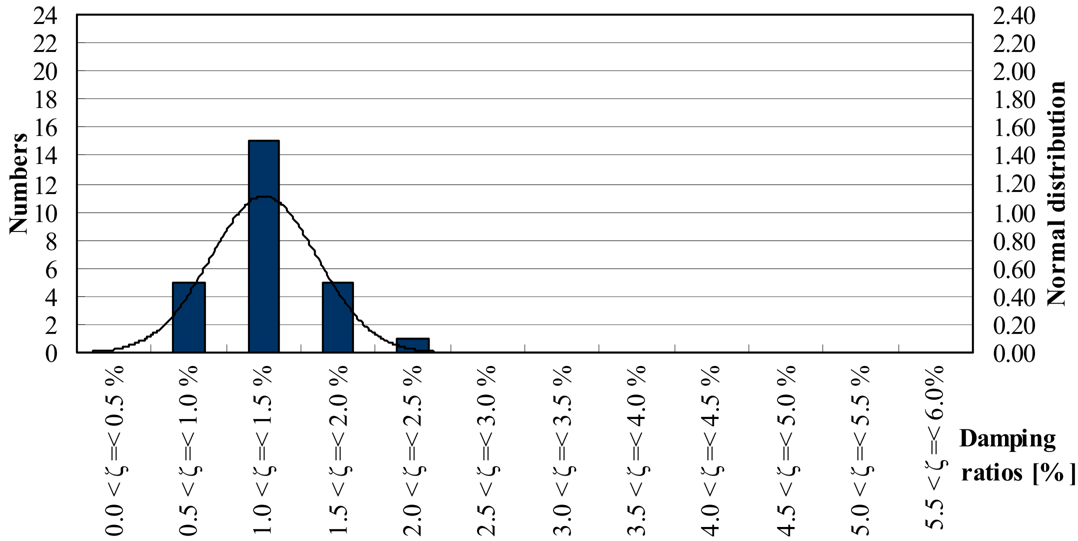
Figure 8.1: Histograms of damping ratios for individual vibration modes of two-side supported I-joist floors



(c) Mode (1,3)



(d) Mode (1,4)



(e) Mode (1,5)

Figure 8.1: Histograms of damping ratios for individual vibration modes of two-side supported I-joist floors (cont.)

For the metal-web joist floors, the mean and 5 percentile damping ratios corresponding to Mode (1,1) were the second lowest among the first five modes. The mean and 5 percentile damping ratios were the lowest for Mode (1,2) and continuously increasing for subsequent modes (Table 8.3). This is a different trend compared to the I-joist floors, with the mean damping ratios of the first three principal modes all below 1% and even below 0.9% for the first two modes.

This analysis reveals that flooring structures built with metal-web joists exhibit far lower damping for the fundamental vibration mode than flooring structures built with I-joists. Also the modal damping corresponding to the two following modes is usually lower. The lower damping of the metal-web joist floors may have been caused by the web material steel.

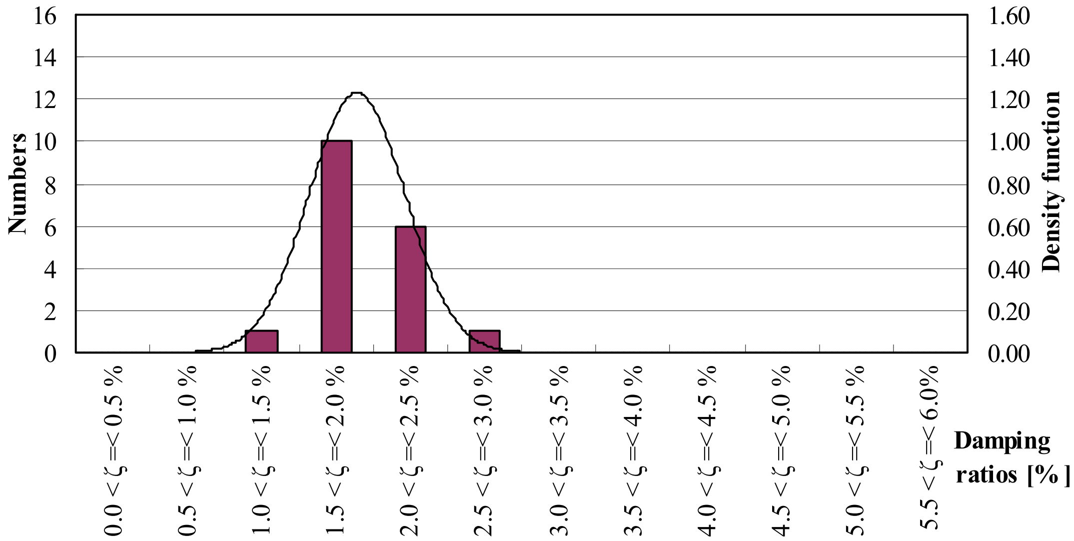
The standard deviation showed very small dispersion of damping ratios especially for the metal-web joist floors although some significant structural modifications were conducted. The standard deviation for the damping of I-joist floors showed higher dispersion in case of Mode (1,1). However, the damping ratios of I-joist floors were in a different range from those of the metal-web joist floors. This all allows for the undertaken distinction between the damping ratios of flooring structures with regard to the different joist types.

Five JJI flooring systems were loaded first by about 30 kg/m^2 and then by another 25 kg/m^2. The original floor mass varied between 17 kg/m^2 - 21 kg/m^2. The minimum, maximum and mean damping ratios for the groups of the five unloaded floors and those with the load increments are shown in Table 8.4 to Table 8.6. The variations in the mean damping ratio due to the imposed mass for the individual modes are presented in Figure 8.4. For the illustration, an averaged mass of 18.67 kg/m^2 was calculated for the grouped unloaded floors.

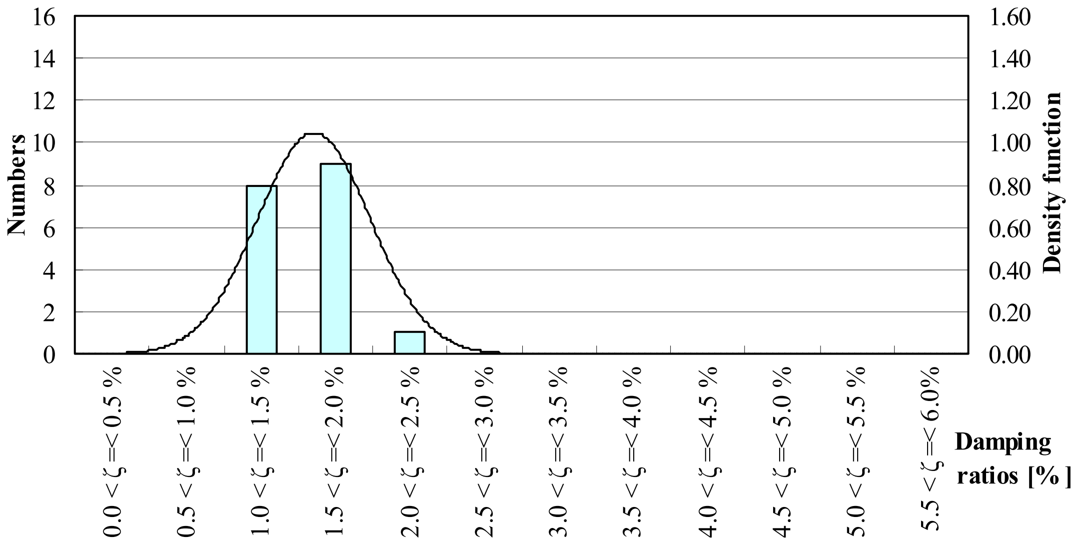
The damping corresponding to Mode (1,1) was largely reduced by the first increment in mass and further reduced to a lower degree by the following increment. The one of Mode (1,2) was slightly raised by the first mass increment and then almost remained the same with only a slight increase for the following mass increment. The damping ratios of Modes (1,3) and (1,4) largely increased. Table 8.6 shows, however, that the standard deviation of Mode (1,3) was relatively high for the second mass increment. Each group comprises no more than five damping ratios, so some outliers can affect the statistical analysis to some extend. However, the general trend should be true by analysing the individual damping ratios and the characteristics in the tables and figure.

Table 8.2: Statistical analysis of damping ratios of four-side supported I-joist floors

ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	-	1.27	1.03	1.08	1.34
Maximum	-	2.55	2.45	2.73	2.25
Mean	**-**	**1.89**	**1.61**	**1.52**	**1.79**
Std Deviation	-	0.32	0.38	0.38	0.28
5 percentile	**-**	**1.49**	**1.07**	**1.10**	**1.38**

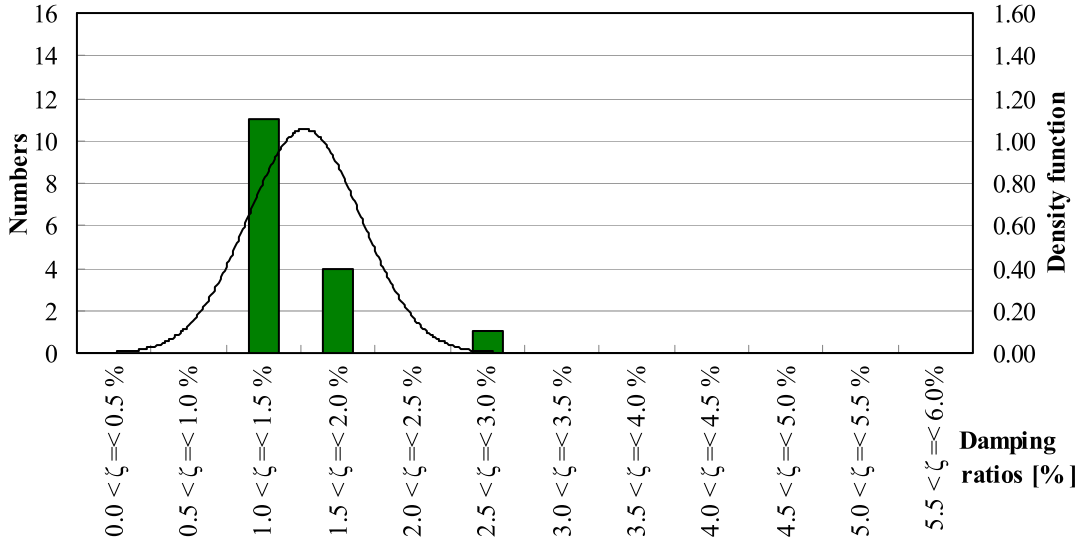


(a) Mode (1,2)

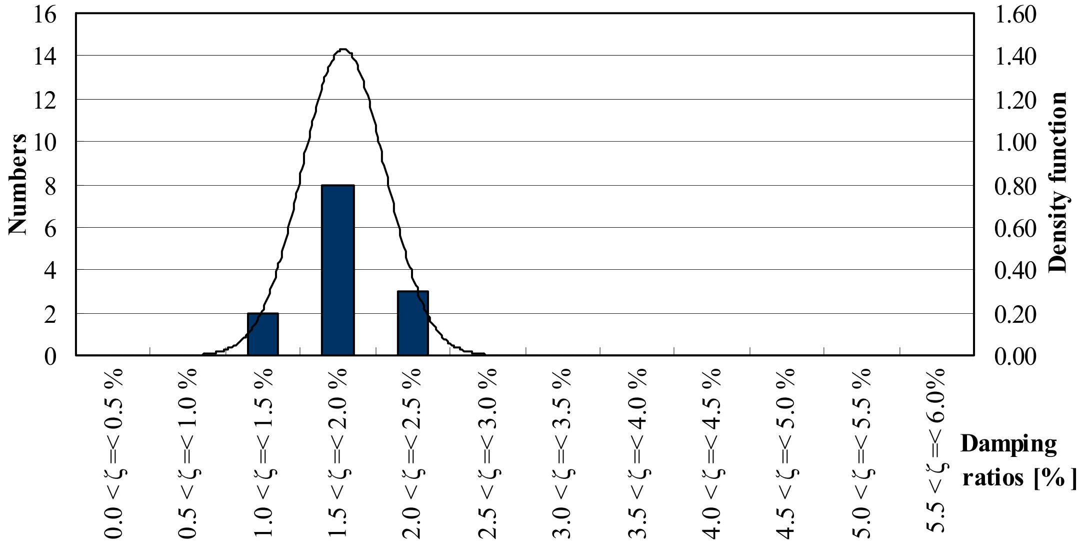


(b) Mode (1,3)

Figure 8.2: Histograms of damping ratios for individual vibration modes of four-side supported I-joist floors



(c) Mode (1,4)

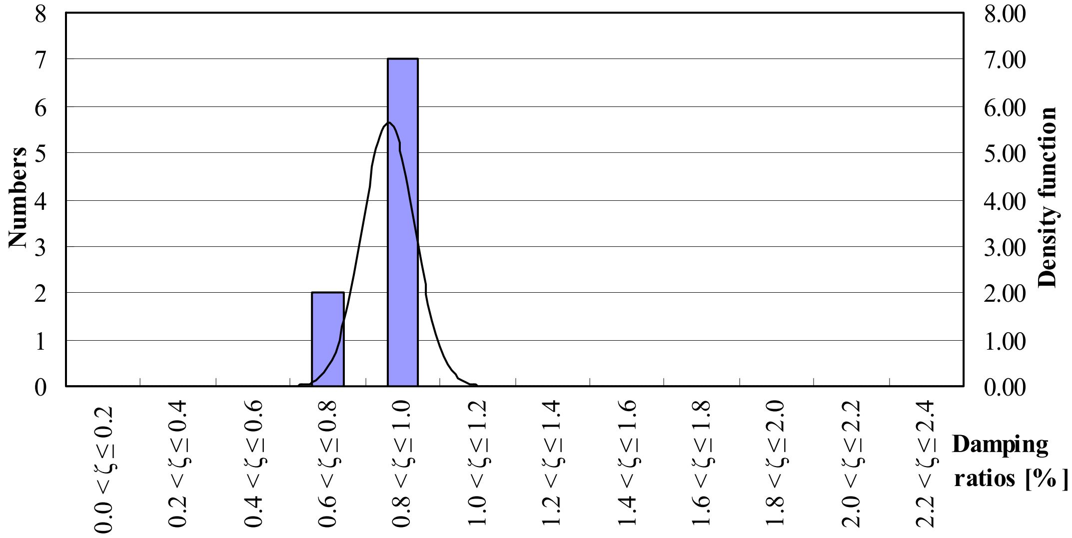


(d) Mode (1,5)

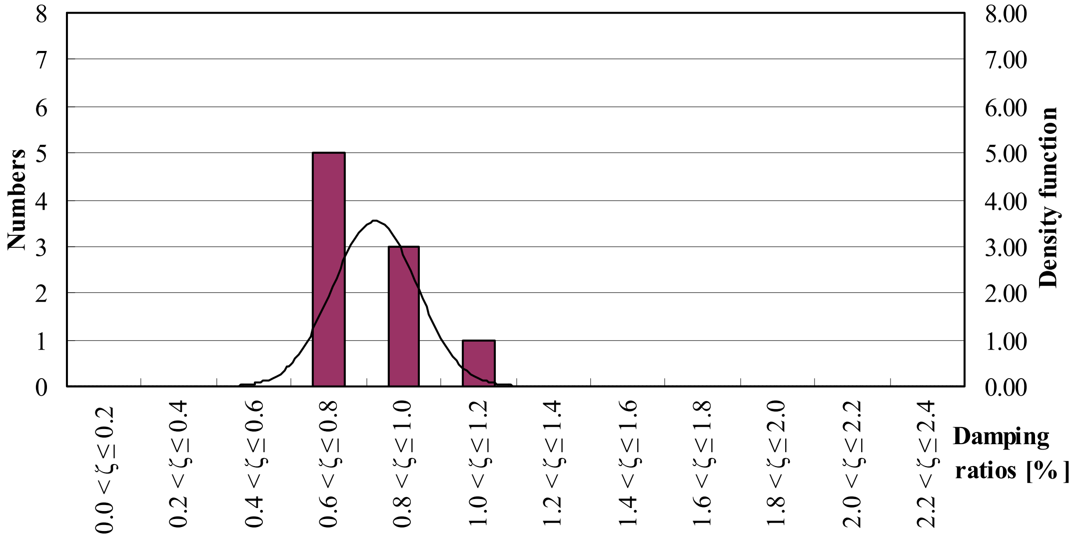
Figure 8.2: Histograms of damping ratios for individual vibration modes of four-side supported I-joist floors (cont.)

Table 8.3: Statistical analysis of damping ratios of metal-web joist floors (two-side supported)

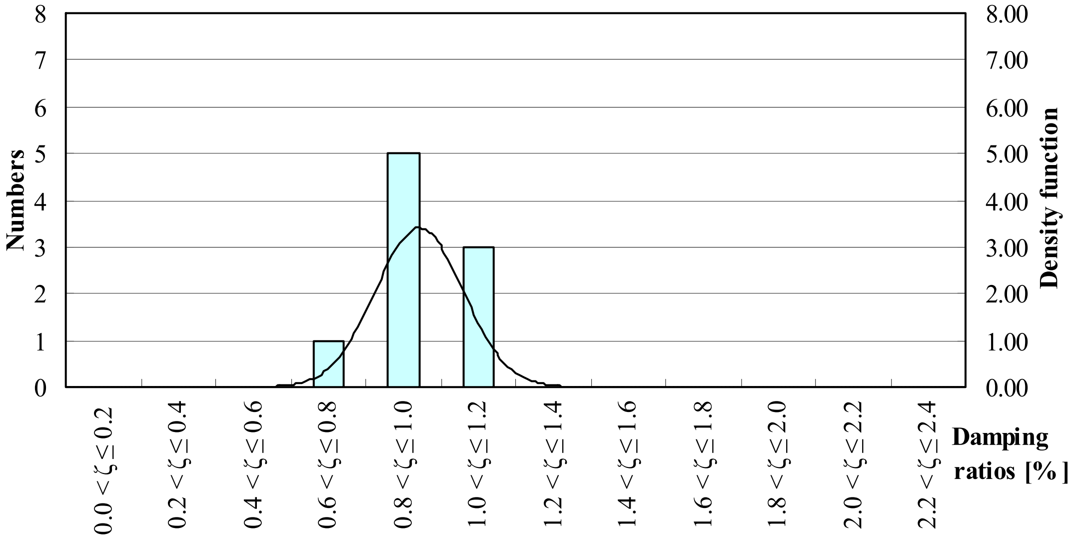
ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	0.77	0.70	0.79	0.88	0.96
Maximum	0.99	1.05	1.13	1.43	1.59
Mean	**0.86**	**0.82**	**0.94**	**1.21**	**1.28**
Std Deviation	0.07	0.11	0.12	0.19	0.20
5 percentile	**0.78**	**0.72**	**0.80**	**0.94**	**0.99**



(a) Mode (1,1)

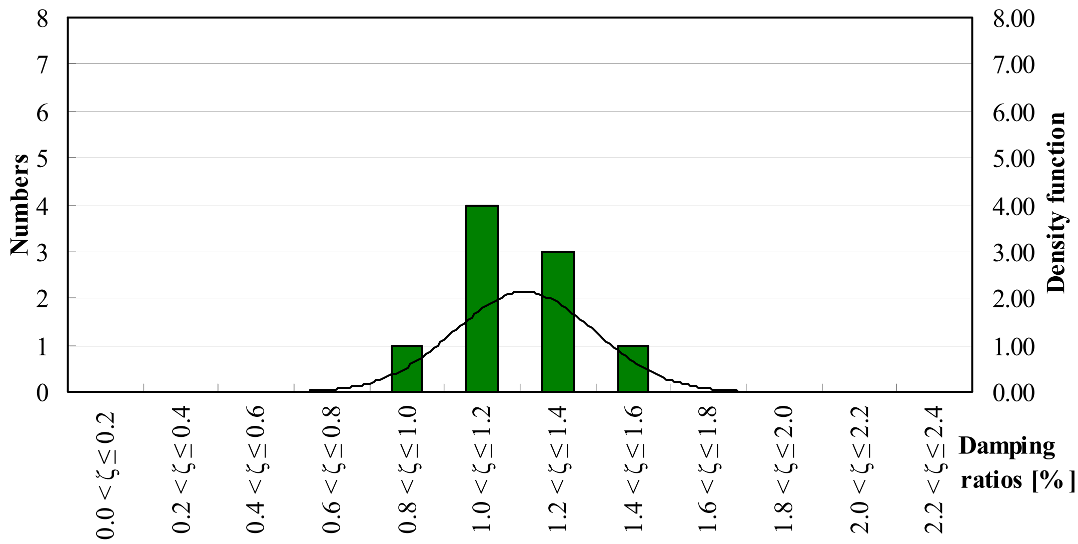


(b) Mode (1,2)

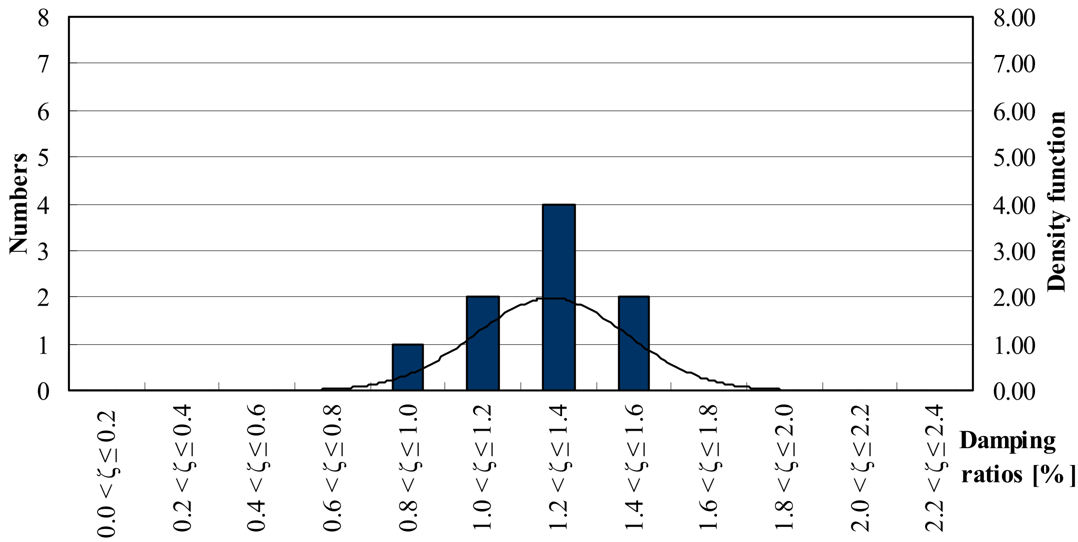


(c) Mode (1,3)

Figure 8.3: Histograms of damping ratios for individual vibration modes of (two-side supported) metal-web joist floors



(d) Mode (1,4)



(e) Mode (1,5)

Figure 8.3: Histograms of damping ratios for individual vibration modes of (two-side supported) metal-web joist floors (cont.)

Table 8.4: Statistical analysis of damping ratios of five unloaded two-side supported I-joist floors selected for investigations with imposed load

ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	2.51	0.95	0.94	0.98	-
Maximum	3.98	1.64	1.08	1.64	-
Mean	**3.36**	**1.13**	**0.99**	**1.19**	**-**
Std Deviation	0.68	0.29	0.06	0.26	-
5 percentile	**2.57**	**0.96**	**0.94**	**1.00**	**-**

Table 8.5: Statistical analysis of damping ratios of two-side supported I-joist floors under an imposed load of 30 kg/m^2

ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	1.25	0.98	1.00	1.25	-
Maximum	1.57	1.59	2.39	3.05	-
Mean	**1.40**	**1.23**	**1.53**	**2.19**	**-**
Std Deviation	0.13	0.25	0.52	0.74	-
5 percentile	**1.26**	**0.99**	**1.07**	**1.40**	**-**

Table 8.6: Statistical analysis of damping ratios of two-side supported I-joist floors under an imposed load of 55 kg/m^2

ζ	(1,1) [%]	(1,2) [%]	(1,3) [%]	(1,4) [%]	(1,5) [%]
Minimum	1.09	1.06	1.24	2.31	-
Maximum	1.43	1.52	4.18	3.28	-
Mean	**1.23**	**1.25**	**3.20**	**2.91**	**-**
Std Deviation	0.14	0.19	1.15	0.37	-
5 percentile	**1.09**	**1.07**	**1.64**	**2.42**	**-**

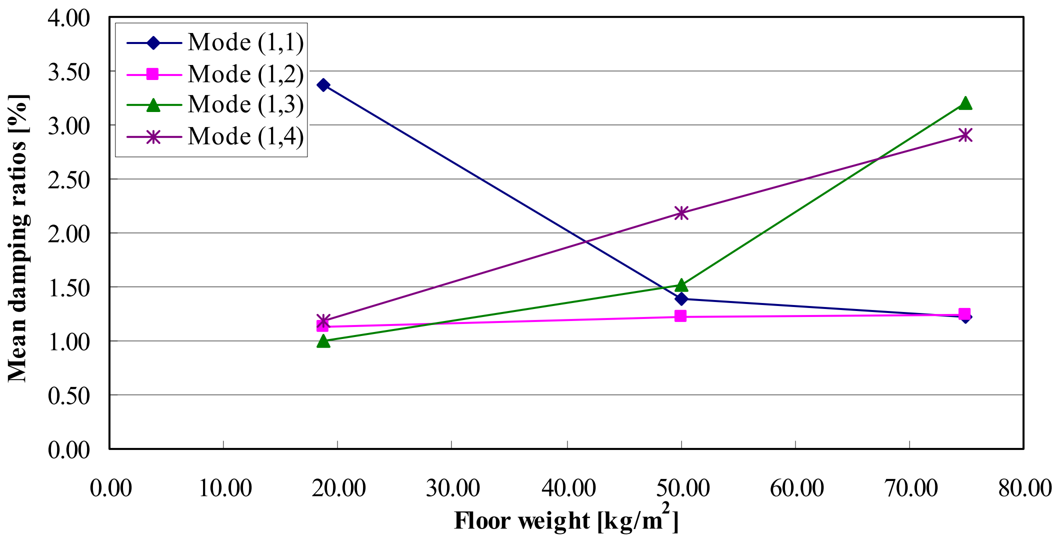


Figure 8.4: Mean damping ratios due to mass increments of first ~30 kg/m^2 and further 25 kg/m^2

As mentioned in the literature review, for a SDOF, the critical damping coefficient c_{cr} is proportional to $\sqrt{k \cdot m}$ and thus ζ is consequently reduced as a result of added dead weight (Ohlsson 1982). This explains the effect of reduced damping regarding the fundamental mode due to the raised mass. The effect appears, however, more complex for higher modes, which revealed counteracting behaviour.

8.3 Summary of the statistical analysis

The main conclusive remarks from the presented analysis can be summarised as follows:

1. The damping corresponding to the fundamental vibration mode of two-side supported flooring systems was on average more than three times higher for floors constructed with I-joists than for floors constructed with metal-web joists.

2. The damping corresponding to the fundamental vibration mode of two-side supported flooring systems constructed with I-joists was on average more than double higher than the damping of the successive modes.

3. The damping corresponding to the fundamental vibration mode of two-side supported flooring systems constructed with metal-web joists was on average similar to the damping of the following two modes.

4. The addition of (imposed) mass significantly reduced the damping corresponding to Mode (1,1) but increased the damping of higher modes, with generally higher effect on subsequent modes.

5. Supporting the I-joist floors along all four sides instead of only along the joist ends highly raised the damping ratio by at least 30% for Modes (1,2) to (1,5), comparing the mean values (for a direct comparison see Section 5.1.1).

6. Based on the standard deviations, the damping ratios corresponding to an individual vibration mode within a specified group varied rather within a limited range.

7. The diversification of damping corresponding to Mode (1,1) was wider than those corresponding to higher modes with respect to the unloaded two-side supported I-joist floors.

8. The diversification of damping corresponding to Modes (1,1) and (1,2) was lower than those corresponding to higher modes with respect to the loaded I-joist floors.

9. The diversification of damping corresponding to an individual mode had a slightly increasing trend for successive modes with regard to the metal-web joist floors.

10. The diversification of damping corresponding to an individual mode was notably lower for the metal-web joist floors than for the I-joist floors.

8.4 Identifying suitable damping ratios for the design

It is controversial how the damping ratio should be used in design. Suggestions were raised by Ohlsson (1982, 1988) and partially adopted by the EC5-1-1. As the dependence of human sensitivity on structural design and corresponding dynamic properties was not investigated in this study, a completely new design approach will not be developed here. However, a proposal of damping ratios appropriate to use in design will be made.

If all vibration modes possessed the same amount of damping, the vibration of higher modes would be damped out faster in relative terms (cf. Eq. (2.29) and Figure 2.3). Furthermore, Ohlsson's tests showed that several principal modes were well excited by walking or heel impacts on floors (Ohlsson 1982). Thus, when the damping of higher modes is much lower compared to the one of the fundamental mode, the corresponding vibration response could become more significant. This is especially likely when the frequency spacing is low. Low frequency spacing could also raise the effect of co-acting modes. The frequency spacing is mainly dependent on floor width and the degree of orthotropy. Typical timber flooring systems as those tested possess a relatively high degree of orthotropy. Some methods to increase the degree of isotropy showed to be successful for raising the frequencies of modes higher than Mode (1,2). The two lowest principal modes were hardly affected. This needs to be considered when selecting a damping ratio for design.

According to the mean and 5 percentile damping ratios of the two-side supported I-joist floors, the fundamental vibration mode possessed highest amount of damping and the one corresponding to the second mode had the lowest amount. As the damping ratio of Mode (1,1) was observed to be much higher than that of Mode (1,2) and as it may be hardly possible to raise the frequency spacing between these two modes for typical timber flooring structures significantly in practice when maintaining the original floor size, the vibration response and thus the damping corresponding to Mode (1,2) becomes more relevant. To be on the safe side, for typical two-side supported timber I-joist floors, the 5 percentile damping ratio of Mode (1,2) should be used for design purposes.

Mode (1,1) was not found experimentally for the four-side supported timber I-joist floors. It is assumed that if it had been found, the corresponding damping ratios would have rather increased compared to the structures supported along two sides. Mode (1,2) possessed the highest damping with respect to the mean values. The lowest was found for Mode (1,4) followed by Mode (1,3). The variation in damping was however insignificant if comparing the mean values between the different modes. The 5 percentile values of Mode (1,3) (the lowest) and Mode (1,4) (the second lowest) were however notably different from the one of Mode (1,2). The frequency spacing between these modes could be raised from certain structural modifications. For typical four-side supported timber I-joist flooring systems, therefore, it should be referred to the 5 percentile of Mode (1,3). When the transverse stiffness is raised due to the use of I-joist blocking (also presumably feasible with similar transverse stiffeners) and thus the spacing between Modes (1,2) and (1,3) appreciably increased, it could be referred to the much higher 5 percentile value of Mode (1,2). The use of I-joist blocking was only investigated on two-side supported floors, but a similar effect would be expected for four-side supported floors.

The same argumentation as above should be valid for the loaded I-joist floors. However, the damping corresponding to the fundamental vibration mode was decreasing while the damping of the second mode was almost unchanged and of higher modes increasing when the floor was loaded. Consequently, the mean and 5 percentile values were still higher for the fundamental vibration mode compared to the second mode for the first load increment and became almost the same for these two modes when the next load increment was applied. It can be expected that another moderate load increment would further decrease the damping ratio corresponding to the fundamental vibration mode so as to eventually become the lowest one. Thus, up to an incremental load of 55 kg/m^2 the damping corresponding to Mode (1,2) should had been decisive. Thereafter, the damping corresponding to the fundamental vibration mode would remain controlling. Hence, the damping ratio to be selected for design would depend on Mode (1,2) until the load level raises the corresponding damping above the one for Mode (1,1). To reduce complexity, it should be reasonable to use the 5 percentile value suggested for the unloaded floors, which would cover well the lowest damping found for the loaded floors and still contain certain safety margin for further reduction in damping due to load charge above the load level investigated. The aspect of raised load, e.g. furnishing, and its effect on the global floor vibration performance is complicated for consideration in any case. Using the damping suggested for unloaded floors is safe and simplifies the procedure.

For the metal-web joist floors, the lowest damping ratios were found for the first two vibration modes. As the damping of the higher mode was slightly lower and the frequency spacing between these modes low, the period of time to damp out the amplitudes of each mode may be similar, assuming similar initial amplitudes. Since the difference was so small, the damping corresponding to the fundamental mode could be used in design.

From literature, the damping determined from measurements on flooring structures in situ is often higher than that found on structures tested in laboratory conditions (e.g. Ohlsson 1982). The environment in a laboratory, however, is more controllable. Also boundary conditions for the floors on site are not always fully clear, e.g. whether a floor in a room is regarded as one unit, or if the whole floor including partitions on top (or below) is defined as the system, etc. Partitions may indeed raise damping of floors. A study by Hu and Tardif (1999), however, showed no variation or even reduction in damping of the fundamental mode for installing different types of partition walls.

The experimental investigations on the loaded floors indicated that load can decrease the damping corresponding to the fundamental vibration mode. The effect of furniture may depend on the character of the object. Furniture with lower inertia than sand bags might contribute to damping by impacting the floor when it is vibrating. Also, humans on flooring structures can add damping as was found by other researchers (e.g. Chui 1987; Lenzen 1966). This effect will not be further considered here. The damping ratios proposed are referring particularly to bare rectangular light-weight timber floors without partitions. However, the values for the I-joist floors cover also the effect of load observed in the experimental study.

Table 8.7 summarises the damping ratios suggested for design. The metal-web joist floors were only tested with supports under the joist ends. The damping ratio for those floors with supports along all edges is still recommended, based on the degree of raised damping found for the I-joist floors.

Table 8.7: Damping ratios suggested for design

Joist type / Supported edges	I-joists	Metal-web joists
Joist ends	0.90%	0.78%
All edges	1.07% (1.49%*)	0.93%

* One row of transverse stiffeners along the whole floor width is installed at mid-span, or more rows at equal divisions in span direction, e.g. the third points.

The low damping ratios suggested for the I-joist floors may appear inadequate compared to the only slightly lower damping values proposed for the metal-web joist floors. In particular, the damping of the fundamental mode of vibration was much higher for I-joist floors. It should be considered, however, that frequency components of higher modes can contribute to annoying vibrations, and may even become dominating as a question of the position of a person on the floor (Alvis 2001; Ljunggren 2006; Ljunggren et al. 2007).

Ohlsson's calculation method for the unit impulse velocity response (Eq. (3.8)) takes several resonances into consideration. The limiting value is based on the product of the fundamental natural frequency and a damping ratio. The damping is assumed to be the same for all modes (Ohlsson 1988). This assumption is proven not to be always valid. A more suitable selection of damping ratios for design depending on distinct structural details and boundary conditions may enhance the evaluation of vibrational floor performances (see also Section 6.5).

The damping ratios suggested in this chapter are based on the flooring structures tested. The standard deviations indicated that the grouping of the damping values to the specified properties was acceptable. The categorised groups and histograms could still be enhanced in future by adding damping ratios from further floor tests when in compliance with the provided criteria or otherwise by extending them, e.g. adding values for floors constructed with other joist types. This could aid to improve the estimation of the 5 percentile values as the number of flooring systems within some groups was not very significant. The suggested design values could then be adjusted.

9. Discussion of experimental findings and analytical studies

The work presented in the Chapters 4 to 8 involved the experimental and analytical investigations of the dynamic response of flooring structures, including modal shapes, frequencies and damping and the deflection under point load. A total of fifty-eight floors constructed with timber I-joists and nine constructed with metal-web joists were tested to identify the effect of structural and non-structural modifications on the dynamic properties. The damping has been investigated further by identifying suitable values for certain conditions so as to support the selection of appropriate damping ratios in design.

Work conducted by other researchers revealed that frequency components of modes above the fundamental one can contribute to disturbing floor vibrations (Ohlsson 1982; Ljunggren 2006; Ljunggren et al. 2007). Alvis (2001) described that other than the fundamental mode could be dominating the severe vibration response sensed by humans. This would depend on the location of the person on the floor as a result of varying locations for the anti-nodes of different vibration modes. These issues have been addressed by these researchers by establishing design criteria considering multi-frequency components, e.g. unit impulse velocity response (Ohlsson 1982, 1988), or by using retrofits for the floors, mainly involving the use of posts underneath (Alvis 2001). The design criteria proposed by Ohlsson (1982, 1988) were sometimes regarded as being too generous (e.g. Ljunggren 2006). The use of posts as retrofits to improve vibration performance may not always be practical.

To account for the above-mentioned issues, practical methods were developed and contribution to enhance the estimation of dynamic properties at the design stage presented. Further effects of varying floor properties on a number of dynamic parameters were detailed

The adopted practical approach to improve design issues is to use double joists at dynamically sensitive locations. This can help not only to raise frequencies of at least the four lowest modes if installed at the appropriate locations, but also to lower the movement at original anti-nodes, possibly as much so as to act as the locations with the lowest vibration displacements for the fundamental vibration mode. Furthermore, the deflection at the sensitive locations can be considerably lowered. The use of double joists at floor centre and at the free edges together of two-side supported floors would usually not narrow the spacing between adjacent natural frequencies but rather widen it, at least for the lower modes, although the degree of anisotropy would be raised.

The unit impulse velocity response is governed by the modal mass, damping and co-action effects from multi-frequency components (Ohlsson 1982). As the double joists raised the modal mass, a positive effect on the velocity response would also be expected although it could not be experimentally verified.

If the same height is maintained for I-joists, an increase in stiffness can be obtained by increasing the flange width or web width. Even though floor structures with equally spaced single joists of the same type may fulfil the requirements for stability, it could be useful from the serviceability aspect to select stiffer joists for the floor centre and free edges. Double joists could be especially effective for this. For existing light-weight flooring structures with unsatisfactory vibrational performance, a very simple option can be to lower the modal amplitude and local static deflection at the identified disturbing locations by just adding extra joists.

It is thus recommended that free edges of light-weight flooring systems be stiffened with double joists if adequate supporting systems cannot be applied at those locations so as to attenuate their contribution to the vibration response. For two-side supported flooring systems, doubling the outer joists at the free edges would lower their modal amplitude, producing modal shapes which approach those of four-side supported floors, at least for the first two principal modes. The use of double joists in floor centre can in particular affect the first vibration mode of light-weight flooring systems in a positive way. Undertaking any or all of these measures may enhance the comfort level, or alternatively identified dynamically sensitive locations can be stiffened accordingly.

Future research needs to focus on the efficiency of double joists on large-scale light-weight floors. For very wide floors, it would be useful to further investigate whether narrowing joist spacing at free edges and in the floor centre could be more efficient while keeping the spacing of the other joists unchanged. This could be achieved in combination with double joists.

To enhance the prediction of dynamic parameters, an approach was undertaken to specify damping ratios with respect to individual vibration modes and construction details. Those damping ratios had been first compared for different floor configurations to identify dependence of damping on (non-)structural detailing and then presented in statistical manner with respect to certain conditions. It is complicated to find a compromise of specifying damping ratios with respect to structural details without being too pedantic as the accuracy for determining damping ratios is limited. The proposed method distinguishes between joist types of the floors and between the numbers of edges supported. Even if the proposed method may not be entirely followed, the presented values from the statistical analysis can aid the selection of damping ratios in design. The proposal given considers contribution of higher modes or their possible dominating roles in disturbing floor vibrations. Hence this does not only rely on the damping of the fundamental vibration mode, but also on the damping of higher modes, e.g. Mode (1,2). This is also because transverse stiffening elements such as I-joist blocking for the I-joist floors or strongback for metal-web joist floors hardly affect the frequencies of the two lowest vibration modes in practice and thus cannot further separate them. Using double joists at the edges in the case of two-side supported floors without stiffening the floor centre, however, would raise the second natural frequency more than the fundamental one, and thus the spacing between them. It is assumed that for four-side supported floors the quarter points could be stiffened with double joists to yield a similar effect.

The spacing between the frequencies of higher modes can indeed be stretched using the aforementioned transverse stiffeners. Another option for raising frequency spacing would be to reduce the floor width. This appears not practical. However, without having this practically reproduced, it may be an option to construct two floor elements of half width instead of one element of full width if frequency spacing is likely to be low. This may, on the other hand, lower the load sharing effect at the locations were the two floor elements meet. Introducing double joists at those locations could attenuate this effect. A continuous deck may still be required to avoid unevenness. This requires further investigations to clearly identify the effects of these measures on the overall vibration performance.

The results from the experimental work also showed that increasing the degree of isotropy by using transverse stiffeners such as blocking or strongback should always be applied to typical timber flooring structures. They reduce point load deflections, which follows from enhanced load distribution. Raising higher frequencies can push some modes outside the frequency range considered critical so that notable contribution of these modes to vibration response could be prevented. With regard to damping capacities it is generally the better option to adopt I-joists instead of metal-web joists.

Using double joists in floor centre and at free edges in combination with transverse stiffeners would be a suitable option to raise all frequencies of first order bending modes, to raise the frequency spacing between the adjacent modes, to lower the number of first order modes below 40 Hz, to enhance the load sharing, to further lower the deflection locally in particular at sensitive floor areas, and to reduce the vibrational movement at dynamically sensitive locations. Doubling the strongback material by using a single element of larger cross section at mid-span or elements of normal size at the third-spans would amplify the effects from using strongback, with a higher efficiency for the former.

Supporting flooring structures along all edges rather than at only the joist ends usually raises all frequencies of first order bending modes with a higher efficiency for higher modes and thus raised frequency spacing. It significantly enhances the damping capacities of all vibration modes. The central point load deflection is little affected on relatively wide floors. Whenever possible, the flooring structures should be supported along all edges. If this is not possible, double joists should be used at the edges, which may not have the exactly same effects but enhances the dynamic behaviour of the floor particularly at these locations.

Spanning the joists in the shorter direction is a favourable option so as to yield higher natural frequencies, higher damping for the lower modes and lower point load deflections. However, the floor width largely influences the distances between neighbouring natural frequencies, and very wide floors can exhibit undesirably low frequency spacing. Spanning the floor in the longer direction may keep larger separations between frequencies of adjacent modes if the same flexural rigidity of the joists is maintained. The aforementioned option of constructing more than one floor element of the same span over the width may be an option of avoiding significant interaction of modes. This however would require further investigations to confirm feasibility and effectiveness.

For I-joist floors, glue should be used in addition to screws to fix the deck to the joists as it should increase the composite action between these materials and therefore raise natural frequencies and lower static point load deflections. The damping corresponding to Mode (1,1) may decrease. As this damping was relatively high for the I-joist floors, this effect may not be significant for those structures. It could become more influential on metal-web joist floors because their damping was found to be rather low. The use of glue at the interface of deck and joists is generally recommended by the NHBC (2008) to reduce squeaking.

A flooring deck which meets the structural and acoustic vibration requirements, considering also limited local deflections at areas between two adjacent joists, should be used. However it should not be oversized since the mass effect can be dominating for the lower natural frequencies and also the damping of the lower modes be reduced. The effect of reducing the deflection can similarly be achieved when using adhesives in addition to screws for fixing a single deck to the joists. Raising the spacing between adjacent natural frequencies can be more efficiently obtained by aforementioned stiffeners.

The addition of equally distributed dead weight considerably reduced the damping of the fundamental vibration mode in the experimental investigations. The addition of a second decking layer yielded a similar effect, which may be explained by the former. However, the damping of the subsequent mode was also clearly lowered, which differs from the trend found for the addition of dead weight. This could be due to that the second layer not only raised the equally distributed mass but also contributed to variations in the structural system and thus to those of the structural damping coefficient.

The selection of deeper joists to alter vibration performance will be influential on structural dimensions. It may be preferable to reduce joist spacing if similar effects are to be obtained. Both measures raise the degree of orthotropy due to an increase in the longitudinal stiffness and hence raise frequencies at the lower modes and reduce point load deflections. Also, local deflections between joists may be more effectively reduced when lowering the joist spacing instead of using deeper joists.

The addition of ceiling as one of many flooring members was investigated on the metal-web joist floors. One of the noteworthy effects by using ceiling is that it attenuates the influence of structural modifications such as the number of modes with raised frequencies or the degree of reduced deflection due to reduced joist spacing. Addition of ceiling otherwise enhances the stiffness in both directions, and hence can lower point load deflections largely and widen the spacing between natural frequencies. The mass of the ceiling is highly influential on the lower natural frequencies, which thus decrease.

The comparison of the measured dynamic performances of metal-web joist floors to those predicted according to EC5-1-1 showed that permanent over-estimation of the fundamental frequency and damping ratio, usual underestimation of the first order modes below 40 Hz, and some underestimation of point load deflections are complicating safe design.

The EC5-1-1 vibration serviceability criteria are not fully accepted in different member countries (e.g. UK and Finland). The unit impulse velocity response is not a popular design criterion, withdrawn in the Finnish NA to EC5-1-1 and bypassed in the UK NA to EC5-1-1 by using a damping ratio doubling the one recommended in EC5-1-1. The high assumed damping ratio in the UK NA to EC5-1-1 likely results in easy fulfilment of the velocity response criterion for typical timber flooring systems and may make this criterion redundant. However, if design is based on the EC5-1-1 but the unit impulse velocity response criterion (indirectly) neglected, the original idea behind the given set of criteria gets violated. Then the vibration in the frequency range above 8 Hz will not be controlled.

10. Finite element analysis to predict natural frequencies, mode shapes and point load deflections of timber I-joist floors

The computer-based finite element method (FEM) has been around for decades. A desire when using the method is mainly to accurately predict the responses of complex systems so as to confirming it to be an adequate substitution of experimental investigations. It can aid to predict the responses of complex structures, which cannot be obtained by simple hand calculations. FEM gained increasing interest in the recent years as appropriate computer programmes became more powerful and the required technology to effectively run analyses of complex systems affordable.

The principal idea of finite element analysis (FEA) is that any complex structure can be divided into a number of (finite) simple elements (the mesh), which are interdependent, to determine the response due to external influences. Each element possesses nodes with degrees of freedom. If nodes of different elements coincide, they interact. A mathematical expression is formulated for the response of each element. All expressions are composed to a set of equations wherein the degrees of freedom are the unknowns. A matrix technique is used to solve the equations (LUSAS 1999b). For detailed background information on the basic principles of FEA it is referred to Henwood and Bonet (1996).

In the study presented in this chapter, the LUSAS FEM software (versions 13/14) was used to develop a finite element model for predicting natural frequencies and corresponding modal shapes of timber flooring systems constructed with I-joists, based on an eigenvalue analysis. The model was also examined with respect to point load deflections to support model verification. The aim was to produce results showing convincing correlation with measured responses by keeping the model rather simple but considering necessary details.

A model is comprised of geometric features to which attributes are assigned. The geometry of the structure is established by selecting coordinates, which define the geometric points. A geometric line in turn is defined by those points, a number of lines can be combined to a surface and a number of surfaces can build a volume (LUSAS 1999b). The geometric points, lines or surfaces can be merged with adjoining geometric elements of the same type, which can then be considered to exhibit full composite action.

The attributes, which need to be assigned, include mesh properties (the finite element types, the mesh refinement/number of finite elements), material properties, geometric properties, boundary conditions, etc. In general, the finer the mesh, the more accurate the results will be but the more time-consuming the analysis. An appropriate compromise of mesh refinement and analysing time needs to be found.

More than 100 different element types are available in LUSAS. They are separated into element groups, such as Beams, Plates, Shells, Joints, etc. For a full list of the element types in LUSAS, see LUSAS (1999a). An element type is selected under consideration of the needs and demands of the model and structural responses under investigation.

The objective of modelling the selected timber I-joist flooring systems is to provide a finite element model for performing reliable eigenproblem analysis. The investigation is further aimed at demonstrating the impact and need of assigning spring stiffness at the supports and at the interface of deck and joists for examining the floor response.

10.1 Modelling timber I-joist floors

The modelling of timber flooring systems has its complexity in the composite nature and anisotropy of the structure and sometimes materials. The basic structure of timber floors is assembled of joists, rim boards, decking sheets and fasteners. In modern construction the joist is composed of different material types with varying dimensions. Timber I-joists are constructed of timber or ply-wood type flanges and usually OSB web. The decking of timber floors consists of a number of adjoining sheets, which thus cause discontinuities in the deck.

10.1.1 Creating the model

The model was created with respect to floors of the JJI test series (see Section 4.2), starting with Floor JJI 2 A. It was found that using shell elements would allow all structural materials to be modelled with the same element type while reproducing accurate material properties, and be suitable for the required eigenvalue analysis of identifying the principal bending modes of the structure.

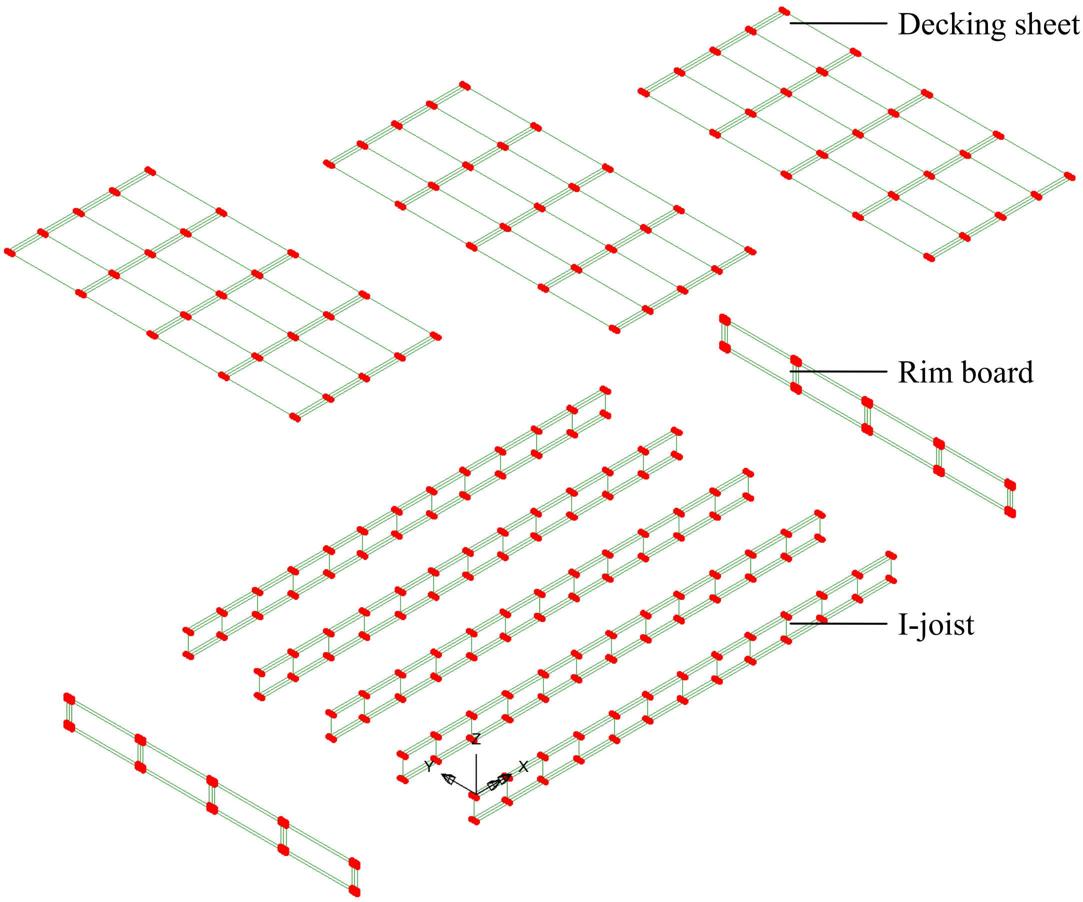
Therefore geometric surfaces were defined for the top and bottom flanges and the web of the joists, the decking sheets and the rim boards. The geometry of the floor was separated with regard to the fastener spacing along the joists and further meshing purposes. The feature of merging geometries was used for the connection of joist flanges and web, and for connecting lines and points at the locations where surfaces were divided for meshing purposes only. The three decking sheets could therefore be created adjacently without being (fully) connected to each other.

Isotropic quadrilateral 8-noded thick shell elements (QTS8) were assigned to the surfaces. In LUSAS, each node of 3D thick shell elements possesses 5 degrees of freedom by default: 3 translations and 2 rotations. An option allows the use of 6 degrees of freedom: 3 translations and 3 rotations, which is automatically enabled for QTS8 elements. The thickness of the material and optionally the possible eccentricity of the nodal plane to the bending plane can be defined in the geometric properties. Eccentricity emerged from the location of the deck and the rim board geometry.

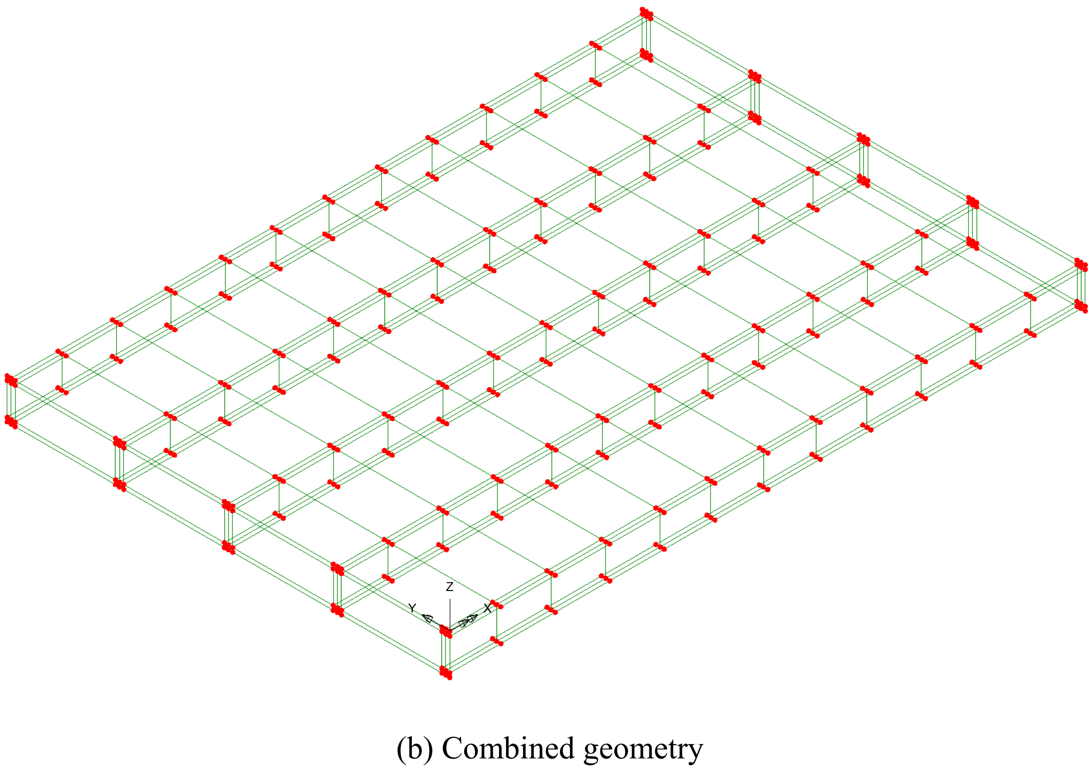
The material attributes, which needed to be assigned, were obtained from literature. The mean values for the density and bending modulus of elasticity were taken from BS EN 12369-1 (2001) for the OSB ($E_{0,mean}$ = 4930 N/mm^2, ρ = 550 kg/m^3) and the P5 particleboard (E_{mean} = 2900 N/mm^2, ρ = 600 kg/m^3), from BS EN 338 (2003) for the C24 solid timber ($E_{0,mean}$ = 11000 N/mm^2, ρ_{mean} = 420 kg/m^3) and from BS EN 1194 (1999) for the rim boards ($E_{0,g,mean}$ = 11600 N/mm^2). A mean density is not given for the latter and thus the one from solid timber with comparable characteristic density value was selected (ρ_{mean} = 460 kg/m^3). A Poisson's ratio of 0.35 was adopted. The geometry of Floor JJI 2 A is illustrated in Figure 10.1 with the individual structural elements (Figure 10.1(a)), in which the geometric features of the same type are fully merged, and with the combined, but not fully merged, floor geometry (Figure 10.1(b)).

The model was established with a number of intermediate steps to figure out to which degree the model could be simplified. This mainly refers to the introduction of spring stiffness at the supports and in form of joint elements between geometries. The first model was created without any spring stiffness, assuming pin-supports with fully restrained translations (free rotation) and full composite action at coinciding geometric points of different geometries, and an eigenvalue analysis carried out.

Then spring stiffness with respect to the translations was introduced at the supports, assigned to each geometric point of the bottom flange ends. The stiffness was first only applied to the lateral y-direction, then only to the x-direction, then only to the vertical z-direction, then only to x/y-directions while movement in any other direction remained fully restrained, and finally to all three directions. For each of those conditions an eigenvalue analysis was performed.



(a) Geometry of the individual structural elements



(b) Combined geometry

Figure 10.1: Created geometry of Floor JJI 2 A

The next step was to implement 2-noded 3D joint elements, which connect two nodes with the same coordinates, at those locations of geometric points, which represented the screws for the connection of decking sheets and joists/rim boards. The selected joint elements (JNT4) have no rotational stiffness. They connect the nodes by springs in the translational directions. Joint stiffness was defined for two degrees of freedom, to allow movement in lateral direction only. A 2D joint element may have been sufficient to fulfil this criterion but was not compatible with the 3D QTS8 elements used. The other coinciding geometric points of deck and joists were unmerged. A similar procedure as for the investigated support spring stiffness was carried out for the connection of deck and joists by applying the appropriate spring stiffness to one direction while keeping the other direction "fixed" (using very high stiffness) and conducting eigenvalue analyses. Since each decking margin was fixed with screws in practice and since also the coinciding geometric points of two adjoining sheets were merged, the spring stiffness assigned to the screws connecting deck and joists had been (manually) doubled at the location where the decking sheets meet. The support conditions were set to be fully restrained regarding translations as only the influence of the added joint elements with varying spring conditions on the natural frequencies was to be investigated. Finally, spring stiffness was assigned to the three translational directions at the supports and to the translational directions in plane of the joint elements at the location of screws fixing the deck to the joists. Connection of the deck to the rim board was only considered at the location of I-joists. The connection of the joists to the rim boards was simplified by merging the coinciding geometric points of these structural elements but still keeping the geometric lines unmerged. The stiffness values of the support springs and joint elements regarding movement in plane direction were assumed to be equal to the slip modulus k_{ser} (BS EN 1995-1-1 2004) and thus calculated from:

$$k_{\mathrm{ser}} = {\rho_{\mathrm{m}}}^{1.5} \frac{d}{23} \tag{10.1}$$

where ρ_{m} is the mean density of the jointed members in [kg/m^3] and d is the fastener diameter in [mm].

The withdrawal stiffness of the fasteners at the supports was first based on the minimum and maximum values reported by Hu et al. (2002), which were determined from experimental investigations on axial load-displacement moduli of fastener-to-wood connections, considering nails and screws.

As the bottom flange ends consisted of three geometric points the support conditions were assigned to, the determined spring stiffness for one screw was multiplied by a factor of 2 to account for the number of screws used per joist end, and then divided by a factor of 3 for distribution onto the geometric points. For establishing the final model, the withdrawal stiffness was then the factor that was used for further adjustments until the prediction matched the measured frequency of the fundamental mode of Floor 2 A. The found factor was then kept for modelling the Floors JJI 2 G, J and JJI 1 A, G, J, considering floors with different joist depths and varying double joists in floor centre.

The number of required finite elements for accurate frequency prediction was determined by running analyses of the initial model with different mesh refinements. The mesh was continuously refined until the difference of the results between two successive analyses with refined meshes became marginal. Then, the one with lower refinement but similar accuracy was selected to save calculation time. The final mesh for the floor model without assigned spring stiffness is shown in Figure 10.2.

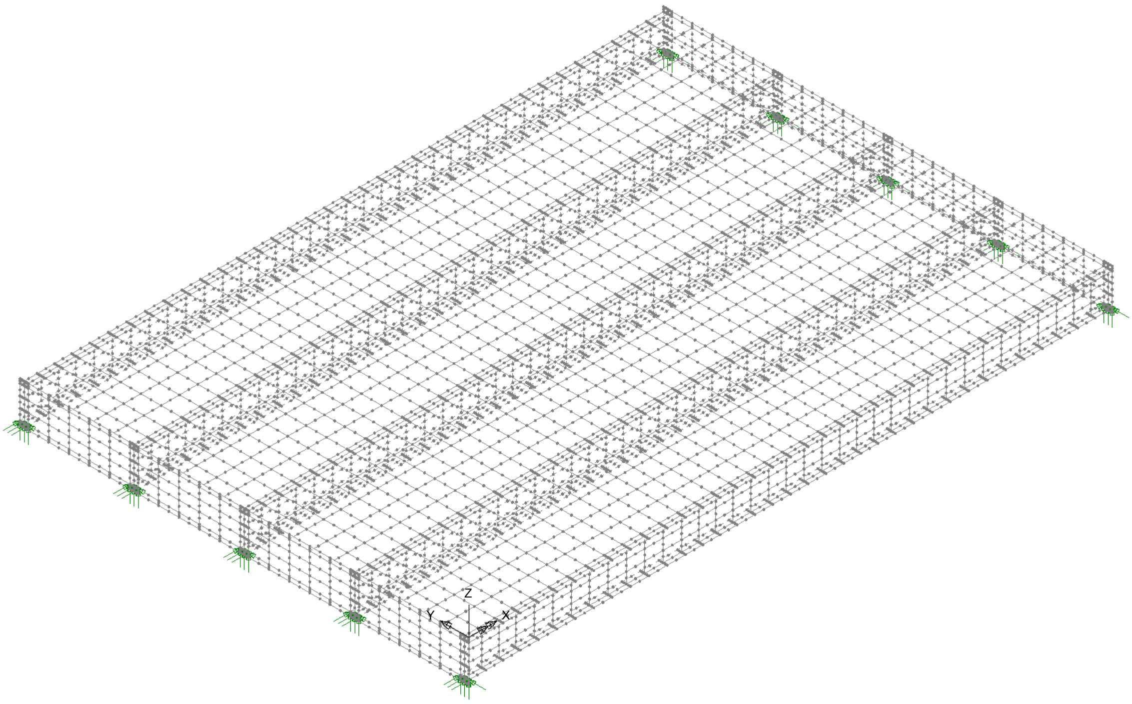


Figure 10.2: Final mesh of model without spring stiffness

10.1.2 Intermediate results

The degree of spring stiffness assigned to supports and joint elements of the initial, intermediate and final models of Floor JJI 2 A is presented in Table 10.1. The accordingly predicted natural frequencies with respect to the first five principal vibration modes are shown in Table 10.2.

Table 10.1: Spring stiffness assigned to supports and joint elements

Floor JJI 2 A	Support			Deck/Joists		
	x	**y**	**z**	**x**	**y**	**z**
	$k_{ser,x}$ [N/m]	$k_{ser,y}$ [N/m]	$k_{withdraw,z}$ [N/m]	$k_{ser,x}$ [N/m]	$k_{ser,y}$ [N/m]	$k_{withdraw,z}$ [N/m]
(A): complete floor without support springs	Fixed	Fixed	Fixed	Rigid	Rigid	Rigid
(B): **(A)** + support y-springs + x/z-fixed	Fixed	1.366×10^6	Fixed	Rigid	Rigid	Rigid
(C): **(A)** + support x-springs + y/z-fixed	1.366×10^6	Fixed	Fixed	Rigid	Rigid	Rigid
(D): **(A)** + support x/y-springs + z-fixed	1.366×10^6	1.366×10^6	Fixed	Rigid	Rigid	Rigid
(E$_1$): **(A)** + support z-springs + x/y-fixed	Fixed	Fixed	3.000×10^5	Rigid	Rigid	Rigid
(E$_2$): **(E$_1$)** + higher spring value	Fixed	Fixed	1.100×10^6	Rigid	Rigid	Rigid
(F$_1$): **(A)** + support springs in x/y/z	1.366×10^6	1.366×10^6	3.000×10^5	Rigid	Rigid	Rigid
(F$_2$): **(F$_1$)** + higher z-spring value	1.366×10^6	1.366×10^6	1.100×10^6	Rigid	Rigid	Rigid
(G): **(A)** + deck screws y-springs + "rigid" x	Fixed	Fixed	Fixed	1.000×10^{17}	1.638×10^6	Rigid
(H): **(A)** + deck screws x-springs + "rigid" y	Fixed	Fixed	Fixed	1.638×10^6	1.000×10^{17}	Rigid
(I): **(A)** + deck screws x/y-springs	Fixed	Fixed	Fixed	1.638×10^6	1.638×10^6	Rigid
(K$_1$): **(A)** + support x/y/z-springs + deck x/y-springs	1.366×10^6	1.366×10^6	3.000×10^5	1.638×10^6	1.638×10^6	Rigid
(K$_2$): **(K$_1$)** + higher z-spring value (support)	1.366×10^6	1.366×10^6	1.100×10^6	1.638×10^6	1.638×10^6	Rigid
(K$_3$): **(K$_2$)** + adjusted z-spring values (support)	1.366×10^6	1.366×10^6	4.500×10^5	1.638×10^6	1.638×10^6	Rigid
(K$_4$): **(K$_3$)** + adjusted z-spring values (support)	1.366×10^6	1.366×10^6	5.500×10^5	1.638×10^6	1.638×10^6	Rigid
(K$_5$): **(K$_4$)** + adjusted z-spring values (support)	1.366×10^6	1.366×10^6	5.250×10^5	1.638×10^6	1.638×10^6	Rigid

Table 10.2: Predicted natural frequencies for varied support/joint properties

Floor JJI 2 A	$f_{(1,1)}$ **[Hz]**	$f_{(1,2)}$ **[Hz]**	$f_{(1,3)}$ **[Hz]**	$f_{(1,4)}$ **[Hz]**	$f_{(1,5)}$ **[Hz]**
(A): complete floor without support springs	34.79	38.34	50.18	68.35	78.98
(B): (**A**) + support y-springs + x/z-fixed	34.79	38.07	49.97	71.37	78.86
(C): (**A**) + support x-springs + y/z-fixed	27.84	31.58	41.11	60.59	74.43
(D): (**A**) + support x/y-springs + z-fixed	27.84	31.51	41.04	61.79	74.39
(E_1): (**A**) + support z-springs + x/y-fixed	24.10	27.04	44.01	67.84	78.88
(E_2): (**E_1**) + higher spring value	30.87	34.48	45.98	67.36	78.63
(F_1): (**A**) + support springs in x/y/z	21.49	24.40	36.62	59.49	73.70
(F_2): (**F_1**) + higher z-spring value	25.70	29.24	38.59	60.37	73.98
(G): (**A**) + deck screws y-springs + "rigid" x	32.30	34.93	43.48	65.06	77.30
(H): (**A**) + deck screws x-springs + "rigid" y	32.23	35.45	42.83	60.35	72.90
(I): (**A**) + deck screws x/y-springs	31.75	34.42	42.27	60.09	70.88
(K_1): (**A**) + support x/y/z-springs + deck x/y-springs	20.69	23.43	33.01	52.91	66.03
(K_2): (**K_1**) + higher z-spring value (support)	24.29	27.53	34.48	53.71	65.86
(K_3): (**K_2**) + adjusted z-spring values (support)	22.18	25.17	33.46	53.27	65.56
(K_4): (**K_3**) + adjusted z-spring values (support)	22.78	25.87	33.70	53.40	65.70
(K_5): (**K_4**) + adjusted z-spring values (support)	22.65	25.71	33.64	53.37	65.68

Applying the calculated spring stiffness values in transverse direction (horizontal y-direction) at the supports did not show much influence on any mode with slightly decreased frequencies, apart from Mode (1,4) with some increase in frequency. Lowering the boundary restrains only in the longitudinal floor direction clearly reduced the natural frequencies. Applying the spring stiffness to both horizontal directions simultaneously thus led to similar results as found for the spring condition in span direction except that the frequency of Mode (1,4) was slightly higher. Using the abovementioned lower and upper limits for the withdrawal stiffness reported by Hu et al. (2002) in two successive analyses showed strongest influence on the frequencies of Modes (1,1) and (1,2), notable influence on Mode (1,3) and little impact on the higher modes in comparison with fixed support conditions. This means that within the range of vertical spring stiffness used for the analysis, the frequencies of the lower vibration modes are dependent also on this stiffness whereas those of the higher modes are hardly. Subsequently, spring stiffness values were assigned to all three translational directions under consideration of the different values in z-direction for two individual runs.

Introducing joint elements at the locations of screws to fix deck to the joists, keeping the vertical direction fully restrained, and setting very high stiffness at one lateral direction and the calculated stiffness at the other one, reduced the frequencies for all modes with the highest effect on Mode (1,3). A noteworthy difference with respect to the degree of the effect was found for Modes (1,4) and (1,5) whether applying spring stiffness in x- or y-direction.

The gradual enhancement of the model showed that spring stiffness needs to be accounted for. Assigning the spring stiffness to the three translational directions at the supports was most effective regarding the lower mode frequencies. The addition of the joint elements to account for spring stiffness with respect to the connection of decking sheets and joists particularly enhanced the estimation of the frequencies corresponding to the higher modes in the final model (see Table 10.1 and Table 10.2).

10.2 Correlation of natural frequencies and modal shapes determined from FEA and measurements

The natural frequencies of the principal modes of the selected floors are presented in Table 10.3 as comparisons of FEA predictions and measured results, together with the differences (absolute values) and errors. The difference between the predicted and measured frequencies was very close with an error of only 0.28 Hz or 1.09% on average for the fundamental vibration mode. The predictions of the frequencies of Mode (1,3) were also successful, with an error of 1.70 Hz or 4.44%. The frequencies of Mode (1,2) were under-predicted by 2.52 to 3.20 Hz, with an average error of 2.87 Hz or 9.72%, and those of Modes (1,4) and (1,5) over-predicted by 5.61 Hz and 5.38 Hz on average, or 11.66% and 9.02%, respectively.

Table 10.3: Comparison of natural frequencies obtained from FEA and measurement

Floor JJI		$f_{(1,1)}$ [Hz]	$f_{(1,2)}$ [Hz]	$f_{(1,3)}$ [Hz]	$f_{(1,4)}$ [Hz]	$f_{(1,5)}$ [Hz]
2 A	FEA Prediction	22.65	25.71	33.64	53.37	65.68
	Measurement	22.66	28.81	34.88	47.65	59.63
	Difference (abs)	**0.01**	**3.10**	**1.24**	**5.72**	**6.05**
	Error (%)	**0.04**	**10.76**	**3.56**	**12.00**	**10.15**
1 A	FEA Prediction	24.44	27.59	36.24	55.45	67.35
	Measurement	24.46	30.11	35.58	47.33	60.72
	Difference (abs)	**0.02**	**2.52**	**0.66**	**8.12**	**6.63**
	Error (%)	**0.08**	**8.37**	**1.85**	**17.16**	**10.92**
2 G	FEA Prediction	24.83	25.76	34.73	53.49	62.79
	Measurement	24.89	28.57	40.17	45.51	57.81
	Difference (abs)	**0.06**	**2.81**	**5.44**	**7.98**	**4.98**
	Error (%)	**0.24**	**9.84**	**13.54**	**17.53**	**8.61**
1 G	FEA Prediction	26.52	27.64	37.65	55.62	64.62
	Measurement	27.29	30.84	39.17	51.38	60.63
	Difference (abs)	**0.77**	**3.20**	**1.52**	**4.24**	**3.99**
	Error (%)	**2.82**	**10.38**	**3.88**	**8.25**	**6.58**
2 J	FEA Prediction	24.15	25.70	34.01	53.42	64.00
	Measurement	23.87	28.58	35.34	49.79	58.85
	Difference (abs)	**0.28**	**2.88**	**1.33**	**3.63**	**5.15**
	Error (%)	**1.17**	**10.08**	**3.76**	**7.29**	**8.75**
1 J	FEA Prediction	25.98	27.58	36.87	55.52	65.88
	Measurement	25.43	30.26	36.86	51.55	60.39
	Difference (abs)	**0.55**	**2.68**	**0.01**	**3.97**	**5.49**
	Error (%)	**2.16**	**8.86**	**0.03**	**7.70**	**9.09**
Mean	**Difference (abs)**	**0.28**	**2.87**	**1.70**	**5.61**	**5.38**
	Error (%)	**1.09**	**9.72**	**4.44**	**11.66**	**9.02**

The FEA detected not only the principal modes of vibration presented. One other mode between Modes (1,2) and (1,3) and some more between the higher principal modes were usually found, mainly including some horizontal movement and rotation with some bending, which were sometimes restricted to local areas or individual elements. These modes may occur either also in reality or due to some simplifications in the model, impacting the options of movement. Also modes of second order (two half sine waves in span direction) were identified. Mode (1,3) appeared twice for Floors 2 G and J whereas the frequency difference between the successive modes was below 0.8 Hz. Since the mode shapes were rather similar but with higher amplitude for the subsequent mode, the higher frequency mode was selected as the principal one. Also Mode (1,4) appeared twice in the analysis of Floor 2 G where the frequency difference was slightly above 0.4 Hz. The mode shapes were almost the same but mirrored along the x-axis. Therefore, the frequency of the first mode to occur was reported in the results.

The FEA produced shapes of the principal modes, which exhibited considerable correlation with those obtained from experimental measurements. The modal shapes of Floors JJI 2 A, G and J are illustrated in Figure 10.3 to Figure 10.5. The modal shapes of the Floors JJI 1 A, G and J were very similar. The presentation of the shapes from the FEA, the deformed mesh, contains more details than the vibration pattern obtained from measurements since the latter are shown as plane figures due to measurements on the top floor surface compared to the full structure illustrated for the former.

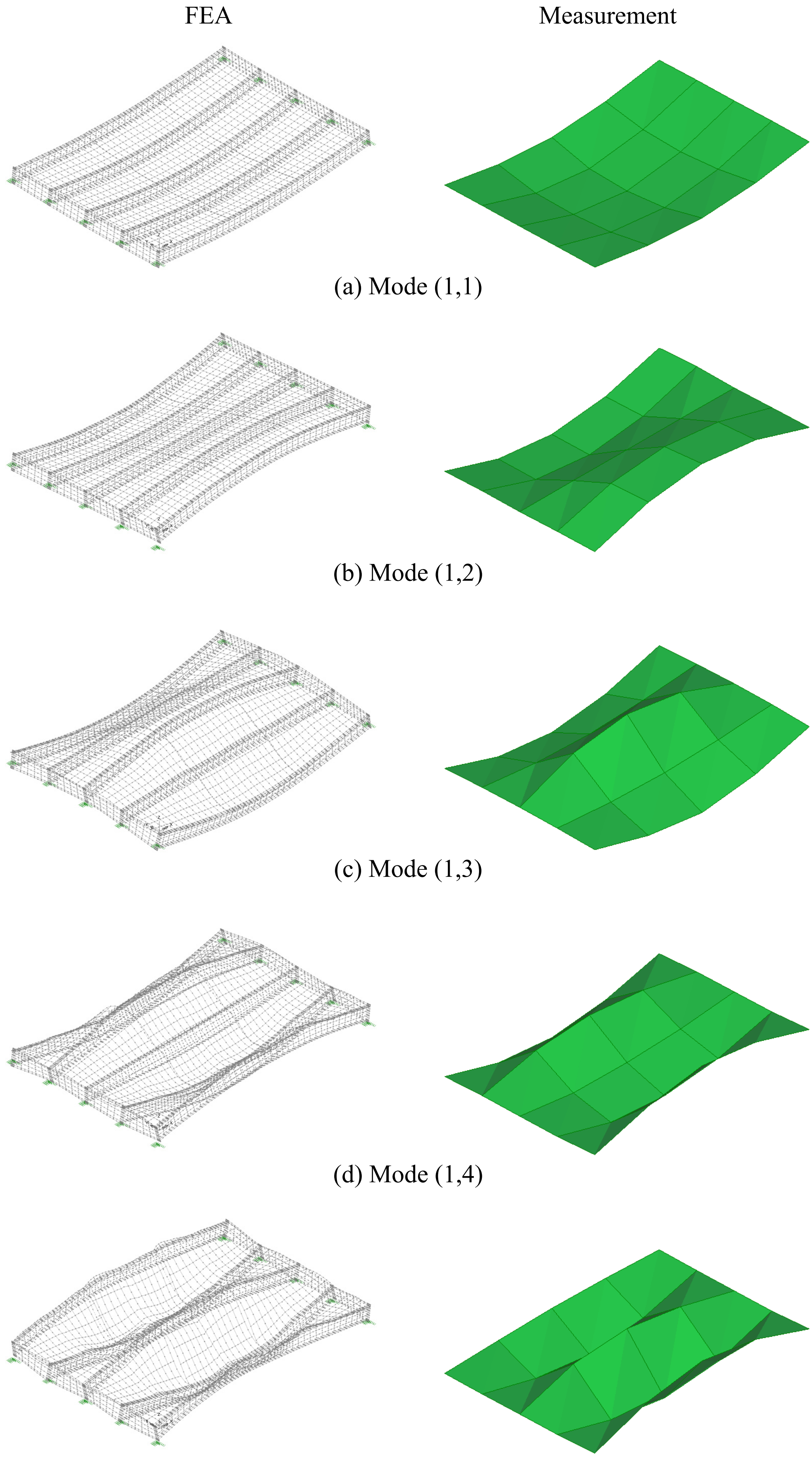
10.3 Examination of variations in mode shapes due to structural modifications

In Chapter 4 it was shown that structural modifications can greatly influence the vibrational shapes of flooring structures. As in Section 5.2.5, it can also be noted from the FEA how the use of double joists alters modal shapes. This is illustrated in (Figure 10.6), which shows the shapes of the odd mode numbers (Modes (1,1), (1,3) and (1,5)) in transverse direction of Floors JJI 2 A and G. This also confirms the findings from the experimental investigations (Section 5.2.5) that the double joists with wider flanges used in floor centre can highly influence the movement at this location, changing the original anti-node of the fundamental vibration mode to the location with lowest movement, and can also clearly lower the relative movement of Modes (1,3) and (1,5) at the floor centre.

10.4 Deflection in floor centre under point load

After establishing the final models of the selected six JJI flooring structures and running eigenvalue analyses, each model was examined with respect to the point load deflection at the floor centre under 1 kN load. The net deflection was obtained by adjusting the deflection value found at the floor centre with consideration of the movement at the joist ends. Figure 10.7 shows the deformed joist mesh of Floor 2 A - other parts were set to be invisible to clearly locate the decisive nodes - including the deflection values in floor centre and at the edges of the central joist ends.

The predicted point load deflections were compared with the deflection results obtained from measurements (Table 10.4) to obtain further verification of the models. The deflections of the Floors 1 A and 2 A were under-predicted, by about 0.2 mm each or 14.5%. The predictions of the other four flooring systems (nearly) matched the measured results. The average error for all floors was 0.09 mm or 7.63%.



(a) Mode (1,1)

(b) Mode (1,2)

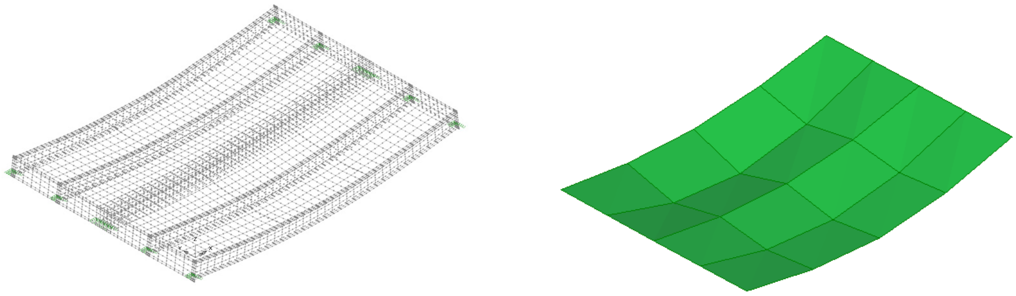
(c) Mode (1,3)

(d) Mode (1,4)

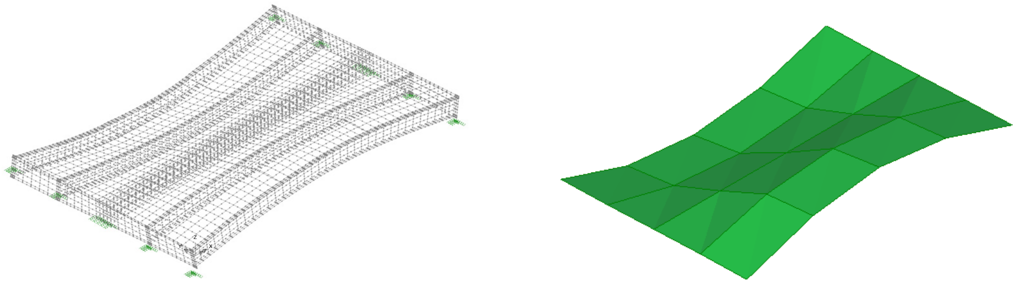
(e) Mode (1,5)

Figure 10.3: Mode shapes of Floor JJI 2 A obtained from FEA and measurements

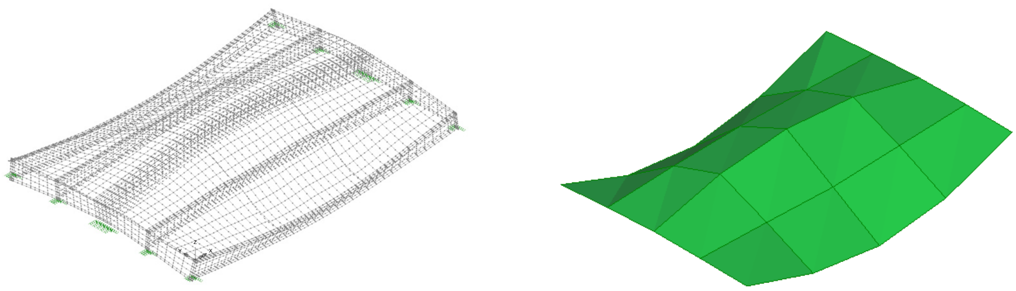
FEA Measurement



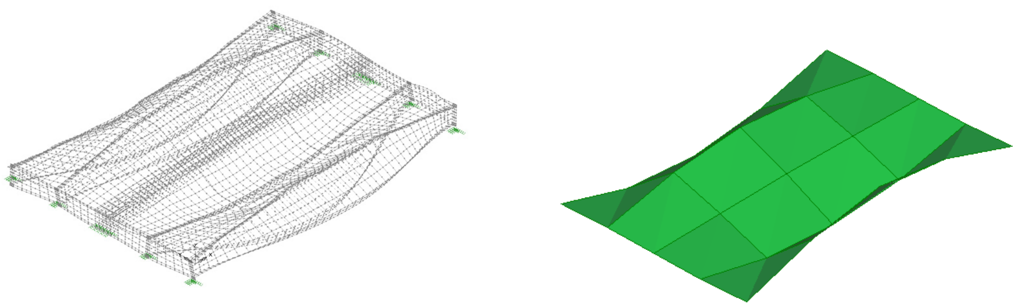
(a) Mode (1,1)



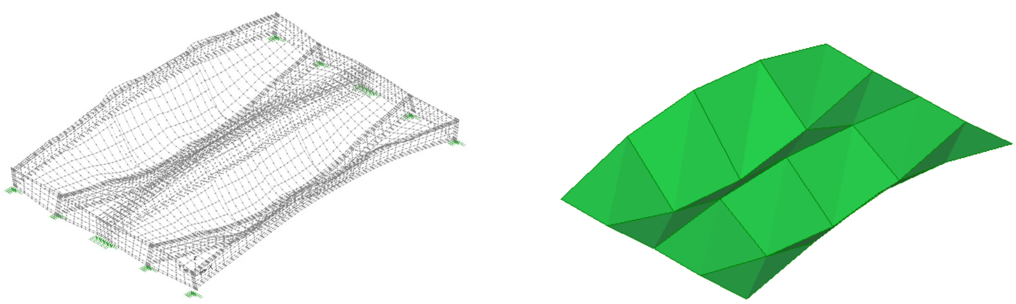
(b) Mode (1,2)



(c) Mode (1,3)



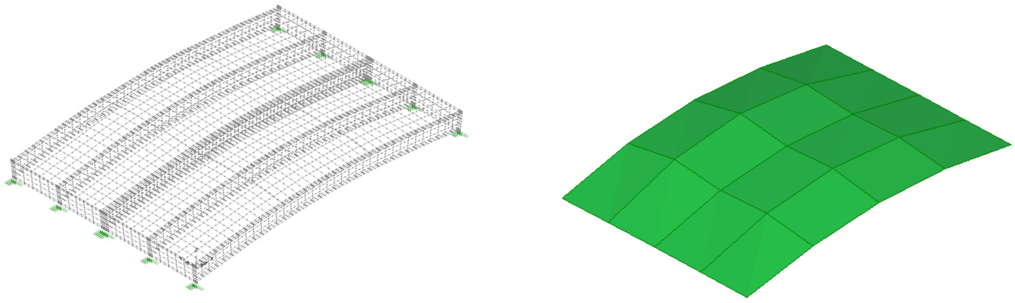
(d) Mode (1,4)



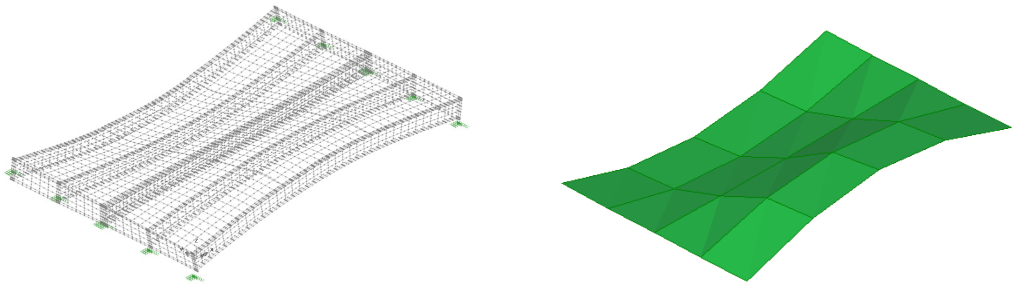
(e) Mode (1,5)

Figure 10.4: Mode shapes of Floor JJI 2 G obtained from FEA and measurements

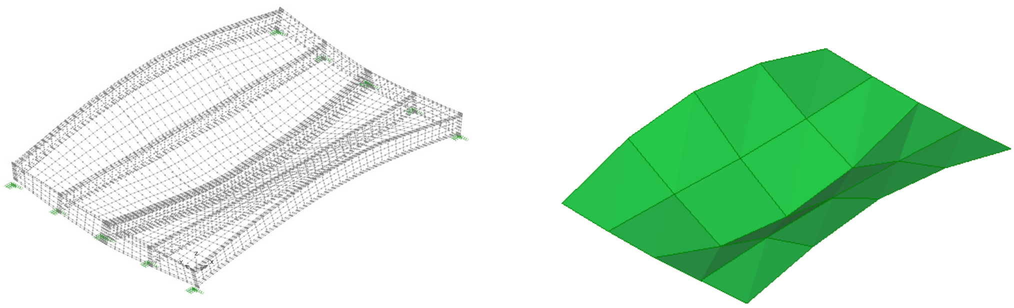
FEA Measurement



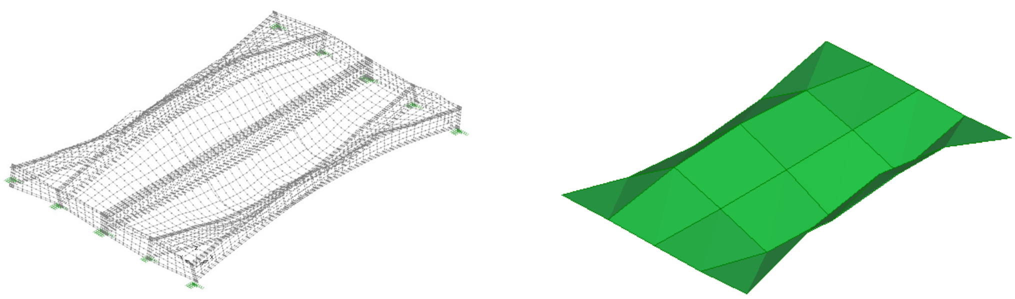
(a) Mode (1,1)



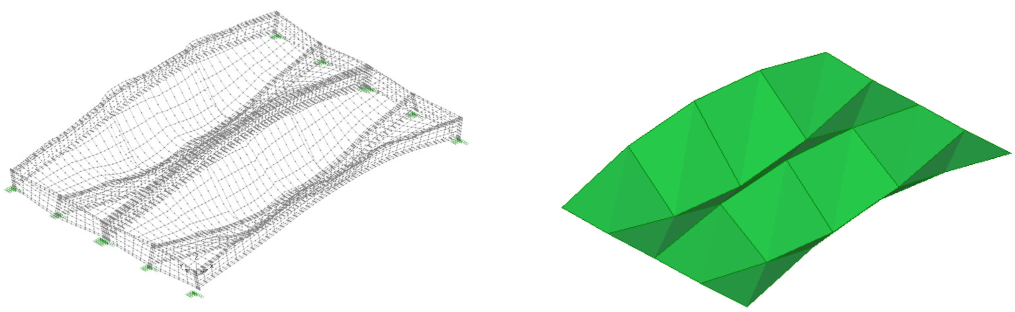
(b) Mode (1,2)



(c) Mode (1,3)



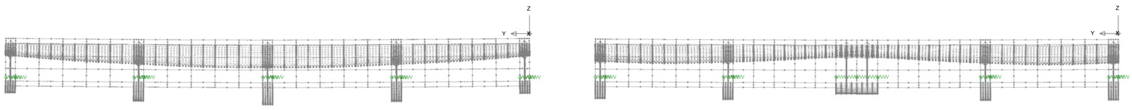
(d) Mode (1,4)



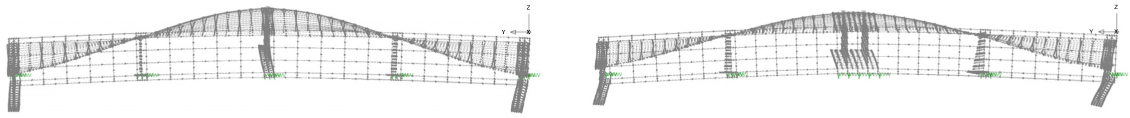
(e) Mode (1,5)

Figure 10.5: Mode shapes of Floor JJI 2 J obtained from FEA and measurements

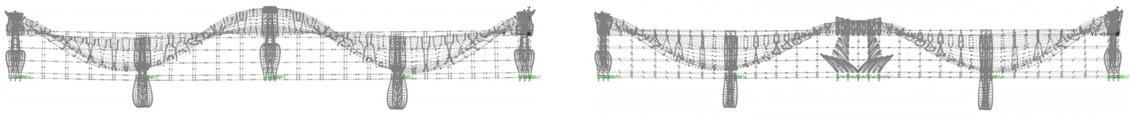
Floor JJI 2 A Floor JJI 2 G



(a) Mode (1,1)



(b) Mode (1,3)



(c) Mode (1,5)

Figure 10.6: Mode shapes in transverse direction obtained from FEA for the first three odd mode numbers of Floors JJI 2 A and G

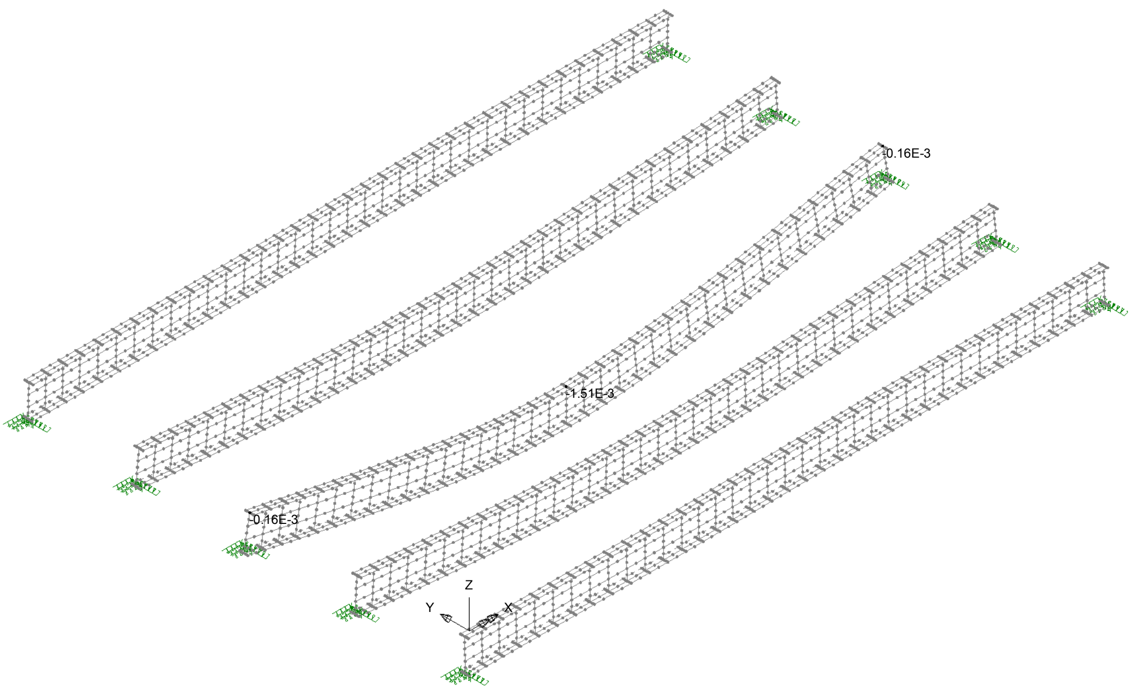


Figure 10.7: I-joists of Floor JJI 2 A under 1 kN point load at the floor centre

Table 10.4: Predicted point load deflections by FEA with the measured ones

Floor JJI		w [mm]
2 A	Prediction	1.36
	Measurement	1.59
	Difference (abs)	**0.23**
	Error (%)	**14.47**
1 A	Prediction	1.12
	Measurement	1.31
	Difference (abs)	**0.19**
	Error (%)	**14.50**
2 G	Prediction	0.57
	Measurement	0.57
	Difference (abs)	**0.00**
	Error (%)	**0.00**
1 G	Prediction	0.46
	Measurement	0.42
	Difference (abs)	**0.04**
	Error (%)	**9.52**
2 J	Prediction	0.90
	Measurement	0.85
	Difference (abs)	**0.05**
	Error (%)	**5.88**
1 J	Prediction	0.73
	Measurement	0.72
	Difference (abs)	**0.01**
	Error (%)	**1.39**
Mean	**Difference (abs)**	**0.09**
	Error (%)	**7.63**

10.5 Discussion and conclusions

This chapter presented the development of a finite element model to be used for an eigenproblem analysis of timber flooring systems constructed with I-joists. Only one element type was used to model the different material properties of the structural timber elements. Joint elements were added to model the connection of decking sheets and I-joists due to the use of screws, which allowed the assignment of spring stiffness. Spring stiffness values were also set for the support conditions. The slip moduli were determined by calculations under consideration of the serviceability aspect. The material stiffness attributes and the withdrawal stiffness were obtained from literature. The stiffness in vertical direction at the supports was the only property that required adjustments. By adjusting the value, basically the withdrawal stiffness of the screws connecting floor joists and supports was to be accounted for. Also the influences due to the stiffness of the supporting structure may be considered by the determined stiffness values in vertical direction.

The determined value for accurate prediction of the fundamental frequency of the base model was then kept constant for the analyses of further flooring structures. The rather accurate prediction of the fundamental frequencies of other selected JJI floors and similar deviations in the prediction of the frequencies corresponding to higher modes confirmed that the selected approach was adequate and reasonable. This was supported by the generally high correlation of predicted and measured point load deflections.

The analysis of the intermediate models demonstrated that consideration of spring stiffness at the supports is required. The use of joint elements at the location of screws to fix the decking sheets to the joists can further enhance the prediction of natural frequencies, in particular those of higher modes. Higher complexity is still required to rebuild the conditions encountered in practice for further raised accuracy in predicting the natural frequencies of higher modes.

The high correlation of the mode shapes obtained form FEA and experimental measurement showed that the composition of the model was successful and that the principal modes mostly can be clearly identified and thus distinguished from all other eigenvalues. The results also allow examining the variation in mode shapes due to structural modifications.

Furthermore, the results suggest that the developed model can be applied for predicting the frequencies of the first three principle modes reasonably. Especially the correlation with the measured frequencies of Modes (1,1) and (1,3) was high in most cases. The under-prediction of the second natural frequency distorts the evaluation of frequency spacing between the first three principal modes.

Therefore, as suggested from the six flooring structures under investigation, the developed model can be applied to get a general overview of dynamic responses of timber I-joist floors, to investigate the variation in modal shapes due to structural modifications and to accurately estimate the fundamental natural frequency, which provides an option for parametric studies when the variation of the fundamental mode of vibration due to structural modifications is to be investigated. Most of this cannot easily and reliably be obtained by hand calculations.

11. Summary

This thesis was focused on the dynamic response of structural timber flooring systems. Basic principles of vibrating systems were summarised first. This was followed by a comprehensive literature review, which concentrated on the sensitivity of humans to vibrations, the influence of floor modifications on the dynamic performance, the control of dynamic floor behaviour, and the predictability of vibration parameters using analytical and numerical methods, so as to describe relevant past research in this field, to show the development of design aspects and to highlight those parameters, which require further consideration. This could be used as the basis for providing a broader understanding of the vibrational serviceability issue.

Extensive experimental and analytical studies were undertaken with support from different industrial companies and the European COST Action E55. Dynamic and static tests on sixty-seven full-scale timber flooring systems with varied structural and non-structural modifications were conducted. Timber I-joists (TJI-joists, JJI-joists) and metal-web joists were used for constructing the floors. The measurements included the determination of the deflection under static loading and of the modal properties of the flooring structures to describe their dynamic behaviour. The frequencies, damping and shapes corresponding to first order modes were of interest. The measured responses were analysed for varied structural configurations. Special attention was paid to locally stiffening dynamically sensitive floor areas so as to attenuate the contribution of various vibration modes to overall floor vibration response. The effect and efficiency of the adopted structural modifications were detailed and recommendations for the enhancement of structural serviceability design were given based on this research's findings. The work furthermore identified aspects, which are suitable for future research. This is fully summarised in Chapter 13.

The damping of the flooring systems was further examined with respect to variation in structural detailing and vibration modes so as to select suitable damping ratios for the design depending on distinct structural properties. The recommendations were then proposed based on joist types and support conditions, accounting also for the effect of wider natural frequency separation when using transverse stiffening elements.

To identify the limitations of current design criteria, the design methods regarding the EC5-1-1 and NAs were examined. The criteria of EC5-1-1, the UK NA to EC5-1-1 and the Finnish NA to EC5-1-1 were investigated within a STSM at VTT - Technical

Research Centre of Finland by assessing the vibration responses of a British I-joist floor, and of a Finnish LVL-joist floor with and without concrete screed. The design criteria of EC5-1-1 and the corresponding UK NA were furthermore examined with respect to the vibrational behaviour of the metal-web joist floors. Several issues which may complicate safe design were highlighted and explained; and recommendations for future work were thus identified.

In order to facilitate an appropriate comparison with analytical results, an FE model was created with the help of the commercial computer-based FE software package LUSAS. The aim was to utilise the model to reliably predict modal shapes and frequencies and also point load deflections of timber I-joist floors. For simplification purposes, only one element type was used for all timber and timber-based materials. The properties of the various material types were adopted from the available literature. The spring stiffness attributes were calculated for the slip-moduli and based on values reported in literature for the withdrawal stiffness. To yield acceptable results, springs were gradually introduced at the supports and at the interfaces between deck and joists in the form of joint elements. The first model rebuilt the base floor for one of the different test series. Further two floors of this series were selected randomly to be modelled subsequently, followed by three comparable flooring systems of the related test series. The predicted results were compared to the measured responses to confirm the applicability of the model.

12. Conclusions

This research yielded numerous results and findings which have highlighted weaknesses in the structural design against, and evaluation of, disturbing vibrations and have led to recommendations for enhanced floor construction and vibration assessment. These all have been presented and discussed in the previous chapters. The key conclusions are summarised as follows.

(1) The current EC5-1-1 design criteria together with the NDPs do not adequately address the issues relating to the dynamic response of timber flooring systems and their associated vibrational problems. Reconsideration of the design criteria is required. This was confirmed by examining the dynamic responses of the metal-web joist floors and by the study within the STSM carried out in Finland. The unit impulse velocity response is unpopular. Neglecting or bypassing it violates the original idea behind the EC5-1-1 criteria if the vibration design is based on them. Fundamental frequencies tend to be overestimated and the number of first order modes below 40 Hz underestimated, which all do not contribute to safe design. Furthermore, the use of more realistic damping ratios may enhance vibration assessment.

(2) The advantages and drawbacks of various structural modifications and of added dead load on the dynamic characteristics of light-weight timber flooring structures have been successfully detailed. Recommendations with respect to structural design for enhancing the dynamic floor performance have been proposed, e.g. glue should be used for I-joist floors in addition to screws for fixing the deck to the joists, I-joists should be adopted instead of metal-web joists to gain higher damping, transverse stiffening members and also double joists at dynamically sensitive locations could be used for effective stiffening, etc.

(3) The use of double joists at dynamically sensitive locations of light-weight timber floors has been adopted, which has yielded promising results to tackle vibration problems and may be easily applied to new and existing structures. Good results may be achieved by the use of double joists at free edges with or without additional double joists in floor centre. The positive effects of this may be amplified by raising the joist width. Raising the degree of orthotropy usually lowers the separation between adjacent natural frequencies. Using the double

joists wisely can raise this separation at least at lower modes although the degree of isotropy is lowered.

(4) In general, structural modifications can yield both positive and negative effects at the same time since the addition or variation of materials affects at least the mass and stiffness of the structure, and sometimes affects the damping notably. The variation in stiffness may then dominate the impact on frequencies at lower modes while the variation in mass may be more influential on the frequencies of higher modes, together with the corresponding side-effects, and vice versa. The use of transverse stiffeners such as strongback for metal-web joist floors and I-joist blocking for I-joist floors, however, may yield no obvious negative effects on typical vibration characteristics.

(5) The width of flooring structures supported along all four edges can significantly affect the spacing between the adjacent natural frequencies.

(6) A phenomenon of practically not finding Mode (1,1) of four-side supported flooring systems has been reported. In future experimental studies the possibility of a missing mode needs to be considered when the vibration response of flooring systems is to be measured. Appropriate measurement locations should be used and a sufficient number of measurement points to detect at least the contribution of the (normally) higher modes.

(7) The current limited information on damping characteristics of light-weight timber flooring systems could be countered by paying high attention to examining damping in detail. Suitable criteria for design, depending on a few distinct structural parameters, have been identified and proposed for bare light-weight timber flooring structures.

(8) Floors constructed with I-joists can exhibit considerably higher damping characteristics than those with metal-web joists while floors fully supported along all four edges can possess much higher damping than floors supported along the joist ends only. Raising dead weight can significantly lower the damping ratio of the fundamental vibration mode but raise the damping of higher modes with an increasing effect for increasing mode number.

(9) The FE model created in LUSAS can be suitably used to get an overview of the most critical natural frequencies of timber I-joist floors and also to perform some

parametric studies with respect to the fundamental frequency, the shapes of the first five principal modes and the static point load deflections. The FE analysis demonstrated the importance of assigning spring stiffness to the supports and to the interface of deck and joists for their connection to obtain more accurate responses. However, the model requires enhancement for predicting the frequencies of higher modes with a higher degree of accuracy.

13. Recommendations for future research

This study identified some areas for future research, including examination of the effect of certain structural modifications, investigations with respect to the design criteria and studies to enhance the predictions using the finite element method. The identified areas and further recommendations are summarised as follows.

(1) Structural modifications

(a) The use of double joists has shown to attenuate the contribution of vibrational modes on the overall vibration response. As this has been tested on structures with limited span and width, it would be of high interest to further investigate the effect and efficiency of this measure on large-scale structures. This method could be modified for relatively wide structures by significantly reducing the joist spacing at the sensitive locations to stiffen them while maintaining the ordinary joist spacing for the other floor areas. This could also be carried out in combination with double joists.

(b) As shown for four-side supported floors, the floor width can have notable influence on the spacing between adjacent natural frequencies. This spacing will reduce for increasing the floor width. Therefore, it may be suitable to produce two floor elements, each of width $B/2$, instead of one element of width B if the spacing between neighbouring natural frequencies is expected to become critical. To identify if this is feasible, experimental investigations are required. The effect of possibly reducing the load sharing at the location where the floor elements meet needs to be considered. Placing double joists at those edges could attenuate the trend. However, continuous decking may be required to avoid unevenness.

(c) I-joist blocking for I-joist floors and strongback for metal-web joist floors showed to be effective in increasing the transverse stiffness of flooring structures so as to enhance their vibrational performances. Other means of increasing the transverse stiffness are available such as engineered floor bridging systems. Future research could help to further identify the most efficient measures of raising transverse flooring stiffness to improve the dynamic floor performances.

(d) There are different possible end conditions for metal web joists such as: built in, top hung or bottom hung. Future research could identify potential variation in dynamic floor responses due to such varied boundary conditions.

(e) Laboratory studies and field investigations could be combined by examining a test structure initially in laboratory conditions before installing it between walls in buildings and retesting it. This could raise the amount of information on the comparability of modal test results obtained from laboratory test structures with real systems, particularly if considering such values in absolute terms.

(f) Inconvenience from vibration reported by occupants may be related to impact sound from the walking of people. For light-weight flooring structures impact sound insulation is an issue especially in the lower frequency range. For measuring impact sound insulation, different test methods exist to simulate footfall excitation. The standardised tapping machine is sometimes replaced by ball dropping or by the use of a Japanese tyre machine to optimise the methods for simulating standardised footfall excitation. The issues of impact sound insulation and suitable testing procedures for evaluating the responses could be further studied in future investigations.

(g) The increased air tightness of buildings is an arising issue. Modern homes tend to be built with a large degree of air tightness to obtain low-energy buildings. In the near future, the testing of air tightness of buildings will become compulsory in Scotland, asking for relatively high demands. As the buildings become more sealed, there will be less air leakage points in the building. If a floor in such a building deflects, the air is squashed and occupants may sense the vibrations. Therefore, in future research, it may be examined whether the increased air tightness will possibly lead to more people being susceptible to vibrations and air movements due to the deflection of floors, which then may result in complaints regarding experienced discomfort.

(2) Design criteria

(a) Parametric studies performed on the unit impulse velocity response criterion of the EC5-1-1 can help to identify if the use of relatively high damping ratios as assumed in the UK NA to EC5-1-1 would generally lead to redundancy of this condition. A high damping ratio used in design could potentially result in an easy fulfilment of the criterion. As a result the original idea behind the given set of criteria is violated and new design criteria would need to be developed or the damping ratio would need to be adjusted if the unit impulse velocity response criterion is otherwise verified as a suitable design parameter.

(b) It will be useful to obtain more damping ratios from measurements on timber I-joist floors and metal-web joist floors, preferably in the same way or in a similar way as in the study described in this thesis. This can help to increase the numbers of grouped damping ratios to determine the values suggested for design, as illustrated in Section 8.4.

(c) The categories of distinct structural details for selecting a suitable damping ratio for design (see Chapter 8) can be enhanced by further sub-categorising joist types, e.g. adding values for solid timber joists. The damping ratios of such structures should be determined in the same way or in a similar way as in the study described in this thesis.

(d) Fundamental natural frequencies of timber flooring systems tend to be over-estimated using the typical design equations for calculations. Solutions for enhancing the degree of accuracy could be sought.

(3) FE analysis

(a) Refinement of the FE model presented in Chapter 10 is required to enhance the prediction of higher natural frequencies, especially those above Mode (1,3). It could be focused on identifying enhanced methods to consider the composite action between deck and joists and on conducting physical tests to more accurately determine stiffness parameters.

(b) The phenomenon of not finding Mode (1,1) on the real test structures may be investigated numerically by modelling four-side supported floors with varying support stiffness, material stiffness and floor aspect ratios so as to study the according implications on Mode (1,1).

Undertaking of any such measures and of the future work as suggested in this thesis could significantly contribute to the improvement of the structural design and the design to EC5 if results are incorporated in future revisions. This would lead to fewer nuisances for residential occupants and enhanced quality of life.

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