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Vibration behaviour of glulam beam-and-deck floors

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A thesis submitted in partial fulfillment of the requirements
For the PhD degree in Civil Engineering

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To my parents, Fereshteh and Javad, and my sister, Maryam

Acknowledgments

Words are not enough to express my sincere gratitude to all the people who provided support during my PhD journey. This thesis would not have been accomplished without the financial assistance of the Canadian Natural Science and Engineering Research Council of Canada and international admission Scholarship awarded by the University of Ottawa.

I owe my deepest gratitude to my supervisor Dr. Ghasan Doudak who has taught me the critical thinking and perseverance to tackle obstacles. His guidance, knowledge, encouragement, trust and providing me professional opportunities helped me grow tremendously.

I am heartily thankful to Professor Emeritus Dr. Ian Smith, an international expert in timber floor vibration, for the time spent to review all chapters meticulously, for his insights and suggestions to improve the thesis.

Besides my supervisor, I would like to thank examiners Dr. Kaveh Arjomandi, Dr. Magdi Mohareb, Dr. Heng-Aik Khoo, Dr. Martin Noel and Dr. Murat Saatuglue for their insightful comments.

I would also like to thank my colleagues, Mrs. Ghazanfarah Hafeez for her guidance, and Mr. Qiuwu Xiao to help conducting parts of the experimental work. Also, I would like to thank all the students and university staffs who participated to conduct the focus group subjective evaluation. I would also like to thank all the members of the structural lab of University of Ottawa, especially Dr. Moslim Majeed for his technical assistance in the lab and understanding. Finally, I acknowledge Mr. Andre Lema from the Western Archrib company, and the Mosaic

building owners (Cuku's Nest Enterprises Ltd) who provided technical data and access to the study building to complete my research for chapter 4.

To my parents, Fereshteh and Javad, and my sister, Maryam: Even though we were miles apart, your continuous love, support, constant encouragement and understanding were main parts of my success. I am so proud to have such a caring and nice family.

Summary

Low-amplitude floor vibrations have become a governing serviceability performance design consideration for floors constructed with low mass-to-stiffness ratio materials such as wood. Studies reported here were conducted at the University of Ottawa to assess vibration serviceability performance of glued-laminated-timber (glulam) beam-and-deck floor systems. Such floors are applicable in non-residential buildings having spans up to about 10 m. The primary goal was to use test and numerical analysis methods to investigate how construction variables (e.g. beam span, beam spacing, addition of nonstructural overlays) affect the vibration responses of such floors. A secondary goal was to assess applicability of vibration serviceability design criteria proposed for other types of floors to glulam beam-and-deck floors. Apart from tests aimed at characterizing responses of laboratory built rectangular plan floors, focus groups were asked to subjective rate acceptability of the performances of those floors. Focus group ratings determined ability of humans to discern alterations in floor motions resulting from construction modifications, based on an opinion survey technique developed by other researchers. This determined that humans can detect and rate performance of floors having different engineering design characteristic, but cast doubts on the consistency of the employed opinion survey technique.

Laboratory tests revealed that mid-span displacements of floors are functions of two-way deflected shapes of floors and are reduced by adding nonstructural overlays and extra beams. Adding non-structural overlays reduces fundamental natural frequency demonstrating gain in modal mass was greater than for modal stiffness. There was inconsistency between the result of

focus group evaluations and predictions of acceptability of floors made using available suggested vibration serviceability design criteria.

Finite Element (FE) models of glulam beam-and-deck floor systems were created and verified using laboratory test data. Based on those models it was concluded that fundamental natural frequencies and mid-span displacements of floors are relatively insensitive to variations in floor width-to-span ratios. However, higher order natural frequencies are strongly affected by floor width-to-span ratios. Increasing thickness of deck elements can decrease natural frequencies and cause them to cluster in ways that amplify surface motions caused by dynamic forces like human footfall impacts.

Field vibration tests were conducted to investigate the dynamic behaviour of a large glulam beam-and-deck office floor having a complex plan shape and support conditions. That floor has long beam spans and partial continuity between bays defined by a mixture of column and wall supports. It was tested before non-structural floor toppings were added and after building completion and occupation. FE modeling of the floor was created and predicted modal characteristics (i.e. mode shapes, natural frequencies) compared with experimentally derived ones. Controlled walking tests were conducted to assess the dynamic response under office occupation conditions. It was concluded the vibration serviceability response of the floor was satisfactory based on peak acceleration measurements and lack of office worker dissatisfaction. Importance of this is the floor has low order natural frequencies less than 8Hz, which means existing proposed design practices created for lightweight timber floor would incorrectly classify

its performance. The discrepancy is indicative that such design practices fail to capture effects of construction variables and damping characteristics of large floors.

In general, vibration characteristics of lightweight floors are highly related to effects of construction details such as plan aspect ratio, boundary conditions and presence of nonstructural elements. Apart from clarifying specifics of how glulam beam-and-deck floors vibrate, this thesis is intended to contribute to Canadian and international efforts to create engineers design methods that robustly predict whether or not specific floors will have adequate vibration serviceability performance under defined floor occupancy conditions.

Contributions to Knowledge

To the best of author's knowledge, thesis is the first comprehensive study of low-amplitude vibration behavior of glulam beam-and-deck floors. Use of diverse laboratory and field test and Finite Element modeling methods has created unique data that can be synthesized in different ways to create new applications and design methods for glulam beam-and-deck floors. Past related research focused heavily on lightweight joisted timber floors. Some recent studies have addressed vibration serviceability of floors constructed using large Cross-Laminated-Timber (CLT) panels, as substitutes for reinforced concrete slabs. Taken together what is reported here and other work creates the foundation for design practices that support applications of wood as an advanced construction material.

Apart from his experimental contributions to knowledge, the author has contributed to development of FE analysis approaches for modelling dynamic responses of glulam beam-and-deck floors. Wood floor structures are very complex and challenging in terms of representation of their structural and nonstructural components and connections that make those components function in unison. Achieved agreement between FE and test derived modal frequencies and mode shapes is believe to be proof of the robust reliability of the FE methods. Specific original contributions the author has made to understanding and development of vibration design practices are the basis of conclusions in Chapter 6.

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List of abbreviations

Ambient Vibration Tests	AVT
American Institute of Steel Construction	AISC
American Society for Testing and Materials	ASTM
Applied Technology Council	ATC
British Standard Institution	BSI
Canadian Standard Association	CSA
Canadian Wood Council	CWC
Enhanced Frequency Domain Decomposition	EFDD
Equivalent Sinusoidal Peak Acceleration	ESPA
Eurocode 5	CEN
Fast Fourier Transform	FFT
Finite Element	FE
Forced Vibration Tests	FVT
Frequency Domain Decomposition	FDD
glued-laminated-timber	glulam
International Standards Organization	ISO
Maximum Transient Vibration Value	MTVV
National Building Code	NBC
National Research Council Canada	NRC
Operational Modal Analysis	OMA
Oriented Strand Boards	OSB
Research and development	R&D
Root Mean Square	R.M.S
Serviceability Limiting States	SLS
simply supported	SS
Single Degree of Freedom	SDOF
Timber Concrete Composite	TCC
UK Timber Research And Development Association	TRADA
Ultimate Limiting States	ULS
Vibration Does Values	VDV
Wood design Manual	WDM

1 Chapter 1

Introduction

Walking or other activities such as jumping might cause low amplitude levels of motion which make floor performance unacceptable to building occupants. Identification and avoidance of construction methods that make floors unacceptable is the goal of design against vibration. In general, floor vibration problems are common especially in floors constructed from materials with low mass to stiffness ratio such as wood (Weckendorf et al. 2015). Inappropriate balance between modal mass and stiffness of floor causes accelerated motions resonate human organs which may lead to adverse experience of floor performance.

Human perception of motion relates to position (laying, walking or sitting), source of vibration (e.g. level of generated vibration in residential floor will be different from a gym or ballroom), application of floor (e.g. acceptable level of vibration in hospital would be different from an office or residential building), and life styles of individuals. Consequently, establishing the criterion to distinguish between acceptable and unacceptable floors is complicated. There is no simple rule of thumb for knowing when a prospective design is correct or incorrect, and each type of floor construction method has to be understood individually. This demonstrates the need for special consideration of vibration design of timber floors.

Vibration serviceability design guidelines provide recommendations engineers can apply in a structural analysis calculation formats. Fundamental natural frequency (f_1), mid-span displacement under 1 kN point load (d_1), unit impulse velocity response (v), and peak acceleration response (a_{peak}) are some of the static and dynamic parameters that are restricted by design guidelines.

However, there exists no general method of performing engineering vibration serviceability design of timber or timber-composite floors that has been proven to be adequate (Weckendorf et al. 2015). This leaves an important gap in the knowledge which the author in part tries to address.

1.1 Thesis Objectives

The objectives of this research were:

- 1- Determine the modulus of elasticity in bending of floor components including glulam beam and deck elements using a dynamic test method.
- 2- Investigate the effect of construction details on vibration responses of glulam beam-and-deck floors.
- 3- Determine whether design formulas suggested by other researches, design codes or design manuals are applicable to glulam beam-and-deck floors.
- 4- Develop and validate a Finite Element (FE) model using the results of experimental work.
- 5- Investigate the effect of floor plan aspect ratio and beam end conditions on vibration performances of large glulam beam-and-deck floors using the verified FE model.
- 6- Investigate the effect of nonstructural elements like concrete overlay and imposed objects such as furniture and humans have on vibration response of a large glulam beam-and-deck floor.

1.2 Scope of Research and Methods

1.2.1 Rectangular floors of limited size

A reference floor with dimensions of 5x5 m was designed based on CSA O86-10 (CSA, 2010). In order to have the floor with maximum bounciness, beams were designed to satisfy the Ultimate Limit State (ULS) but not Serviceability Limit State (SLS) criterion in Canada (CSA 2014). To investigate the effect of beam span, increasing the number of glulam beams, adding intermediate beam support as well as adding nonstructural overlay, additional eight floors were constructed and investigated. Measurements of d_1 , low order modal frequencies and the viscous damping ratio for the fundamental mode were undertaken. Focus group subjective opinions of how constructed floors respond to walking footfall impacts were collected. Measured values as well as subjective test results were compared with available vibration serviceability design criteria to determine whether such criteria can be applied to vibration serviceability design of glulam beam-and-deck floors.

The floors were modeled using the commercial FE software Abaqus 6.12 (SIMULIA, 2012), with input from laboratory measured component properties. After verification, the FE model approach was used to assess effects of construction details such as boundary conditions and plan aspect ratio have on vibration performance of floors. The ball drop test data was used to investigate if models accurately predict time-history response of floors subjected to impacts.

Analytical formulas were used to predict f_1 and d_1 values. The d_1 formula was developed from equivalent orthotropic plate theory after Chui (2002).

1.2.2 Large complex floor

In-situ vibration tests were carried out on a large glulam beam-and-deck floor to determine its low order modal frequencies and damping. Modal frequencies were compared to those obtained using analytical formulas and FE models.

1.3 People perception of vibration

Although the response of the people to floor motion was not a main objective of this research, it is of interest to the overall understanding of the topic. Wiss and Parmelee (1974) proposed a vibration criteria to rate the level of people's perception based on the amplitude, frequency and damping of the applied transient wave representative of footfall impacts on floors. An electromagnetic shaker was incorporated into the center of the floor and a total of 202 signals with different frequencies, peak displacement and damping ratios were applied to each subject. The wave form used to conduct the test is shown in Figure 1.1:

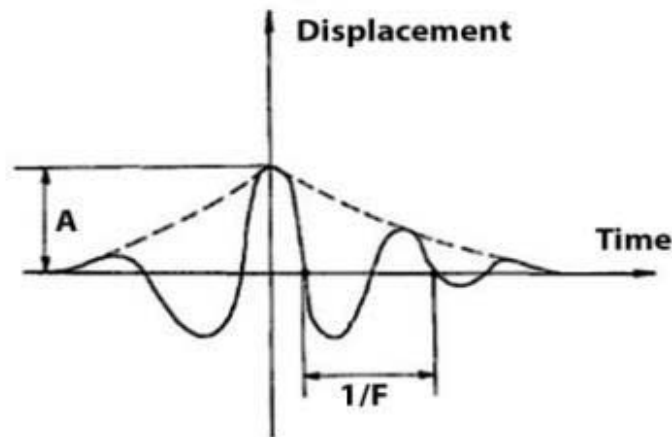


Fig. 1.1. Transient vibration applied in (Wiss and Parmelee 1974)

The individuals were asked to rate their perception from 1 to 5 as: 1= imperceptible, 2= barely perceptible, 3=distinctly perceptible, 4= strongly perceptible and 5= severe. Statistical analysis was performed on the gathered data and the following formula was suggested to rate the human response in damped vibration:

$$R = 5.08 \left(\frac{FA}{D^{9.217}} \right)^{0.265} \quad (1.1)$$

Where f is frequency in (Hz), A is displacement in (inch), D is damping ratio and R is response rating. For an undamped wave one obtains:

$$R = 6.82 (FA)^{0.24} \quad (1.2)$$

The study concluded that by increasing the damping ratio, the perceptibility of transient vibration with certain frequency and maximum amplitude would progressively decrease.

As mentioned earlier, the acceptable level of vibration for human highly depends on the life style, activity and position. Current ISO standards including ISO 2631-1 (ISO 1997), ISO 2631-2 (ISO 2003) and ISO 10137 (ISO 2007) consider acceleration as the quantity of vibration magnitude. ISO 2631-1 (ISO 1997) provides general guidelines on the evaluation of human exposure to whole body vibration. Human body sensitivity is frequency dependent and therefore the measured acceleration needs to be frequency weighted (Griffin 1997). This standard provides weighting functions for cases where exposure of the people to vibration is determined (Fig. 1.1). ISO 2631-2 (ISO 2003) is an extension of ISO 2631-1 (ISO 1997) and provides guidelines for human exposure to vibration in buildings in the frequency range between 1 and 80 Hz. Also, it provides weighting function for cases where the body exposure to vibration is not determined in the building. The issue with ISO 2631-2 (ISO 2003) is that it does not provide any acceptable limit for vibration. To overcome this deficiency, ISO 10137 (ISO 2007) provides base curve and multiplying factors

related to each floor type application (i.e office, residential buildings and workshops). The ISO base curve and the table including multiplying factors is shown in Figs. 1.2 and 1.3.

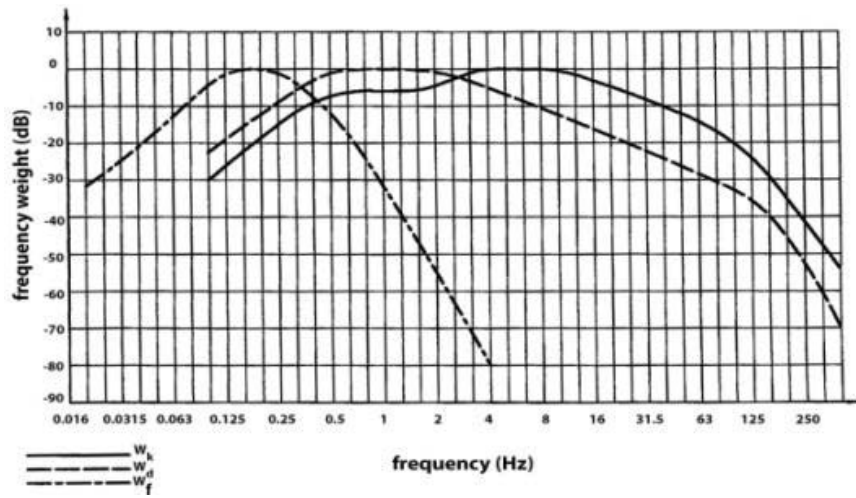


Fig. 1.2. Frequency weighting curves for weighting (ISO 2631-1, 1997)

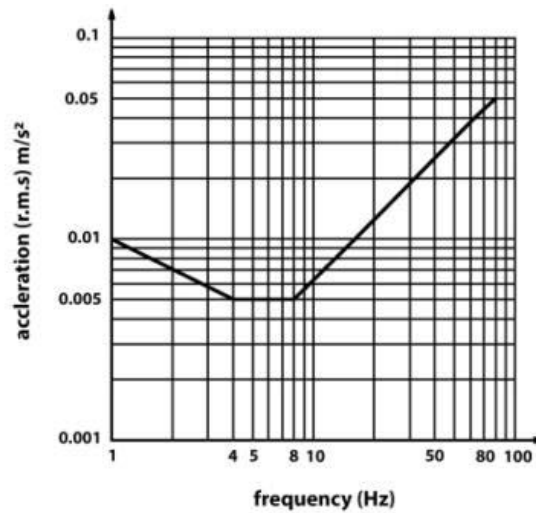


Fig. 1.3. ISO base curve for building vibration along Z-axis (ISO 10137, 2007)

Table 1.1. Factors used to specify satisfactory magnitude of human response to building vibration (ISO 10137, 2007)

Place	Time	Multiplying factors for base curve	
		Continuous vibration and intermittent vibration	Impulsive vibration exciation with several occurances per day
Critical working areas (e.g some hospital –theather, some precision laboratories, etc)	Day	1	1
	Night	1	1
Residential (e.g flats, homes, hospitals)	Day	2 to 4	30 to 90
	Night	1.4	1.4 to 20
Quiet office, Open plan	Day	2	60 to 128
	Night	2	60 to 128
General office (e.g. schools, offices)	Day	4	60 to 128
	Night	4	60 to 128
Workshop	Day	8	90 to 128
	Night	8	90 to 128

Ljunggren et al (Ljunggren et al 2007) conducted three different studies to assess human perception of vibration. Their test setup contained shaker which was connected to a wooden plate representative of the floor surface. The tests consisted of applying vertical vibration with single frequency content to determine the human perception threshold values as well as applying vertical vibration with a base component of 8 Hz accompanied by another signal with different frequency content to determine the perception threshold of a signal with dual frequency content. The result of the first test agrees well with other standards such as the ISO base curve. The result of the second study showed that the threshold value is higher in dual signal except in signal with closely spaced modes. The study also concluded that the annoyance increased with reduced spacing between the frequency of second signal relative to the base and becomes worse by increasing the amplitude. In general, the result of the study showed that modal clustering is an important parameter which needs to be considered in vibration design of floors with closely spaced modes. This supports previous

observation that modal clustering attenuates the dynamic response of floors. (Smith and Chui, 1988; Filiatrault et al 1990).

1.4 Literature on Vibration Behaviour of Timber Floors

1.4.1 Classification of floors

Human footfall impacts have low and high frequency components, with low frequencies being in the order of 0-8 Hz and frequency component in the order of 8-40 Hz. Consequently, it has been suggested floors with frequencies below and above 8Hz can be defined as low and high frequency floors respectively (Ohlsson 1982, 1988). Other researchers and reference documents suggest other frequencies as separating high and low frequency floors, ranging from 7-10 Hz (BSI 2008, ISO 2007, Murray et al 1997). Irrespective of the definition of an associated frequency range the concept is low frequency floors will resonate when humans transit them. A footfall forcing frequency/frequencies (or a harmonic of it/them) corresponding to f_1 , another low order modal frequency (or sub-harmonics of it/them), is cited as the primary concern related to motion that might irritate or annoy human building occupants. Resonance results in responses with higher amplitudes that can persist beyond human pacing periods (Ohlsson 1982, 1988). In practical terms, floor responses to a train of footfall impacts associated with normal activities do not decay rapidly enough to have negligible effects on serviceability of a floor. Low frequency components of walking loads can generate oscillatory motions with semi-static amplitude, which mainly relates to stiffness of a floor (Ohlsson 1982, 1988). Ohlsson (1982, 1988) reports high frequency components of footfall impacts can excite floor modes with frequencies up to 40 Hz.

1.4.2 Vibration design of high frequency timber floors ($f_t > 8$ Hz)

1.4.2.1 Part 9: NBC and other Canadian design methods for joisted floors

The following is based on the report by Onysko et al. (2000) of traditional Canadian approaches to avoiding vibration serviceability problems with lightweight sawn lumber joisted timber floors located in houses and other small buildings. They report the traditional criterion limited the maximum displacement under a design live load of 1.9 kPa to span divided by 360 ($L/360$). Joists were assumed loaded in proportion to their spacing, with no composite action between joists and subflooring. Consequently, each joist could be designed as a simple beam. When new materials such as plywood or Oriented Strand-Board (OSB) sub-flooring, engineered wood joists and new construction details like nailing and gluing sub-flooring to joists entered the scene (used in lieu of sawn lumber joists and floorboards), the allowable displacement limit was changed to limits like span divided by 480 or 600. The Canadian Eastern Forest Products Laboratory (now FPInnovations) developed an alternative criterion based on the maximum displacement of a floor under 1 kN concentrated load (d_1). For floors with span < 3 m the displacement limit was 2 mm, while for other span $d_1 \leq 7.217/\text{span}^{1.274}$ (mm), with span in m. Another revised criterion of $d_1 \leq 8/\text{span}^{1.3}$ (mm) was accepted for derivation of prescriptive maximum spans of sawn lumber joists defined in the 1990 edition of Part 9 of the Canadian National Building Code (known colloquially as the NBC). Fig.1.4 compares old and new criteria. Variations on the 1990 vintage Part 9: NBC approach were developed by the Canadian Construction Material Centre for engineered wood product joist having relatively large spans (CWC 1997).

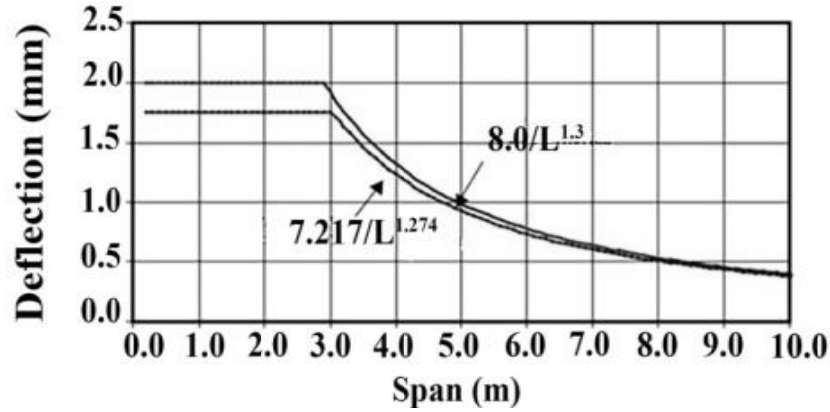


Fig. 1.4. The original and modified criterion for NBC (Onysko et al 2000)

Displacement limit methods mentioned above were initially based on long standing practical experience of what maximum joist spans did not result in building occupant dissatisfaction with how floors performed. The later vintage $d_1 \leq X/\text{span}^Y$ approaches were based on field surveys data correlating d_1 with joist span and how building occupants subjectively assessed performances of floors during normal use conditions. There is no attempt with such approaches to understand the science of the situation.

1.4.2.2 Dolan et al.

Dolan et al., (1999) proposed a criterion based on limiting the f_1 of floors to specific values based on dynamic testing and subjective evaluations of laboratory built floors, and floors in unoccupied and occupied buildings. Tested floors comprised of double-tee beam specimens with 2.7-8 m length. Applied sheathing was mostly oriented strand board (OSB), which was nailed or glued to the joists. Heel drop test was performed to excite floors. Floors were classified as having acceptable, marginal or unacceptable performances. The authors proposed that f_1 for acceptable

floors in unoccupied and occupied buildings need to be >15 Hz and >14 Hz, respectively. They also suggested a formula for estimating f_1 . The composite action between joist and sheathing was neglected in the proposed formula. Related work by Patrick (1997) showed when f_1 is greater than 15 Hz, displacement of a single joist under its own weight would be less than 1.4 mm. This is not surprising because the fundamental natural frequency and self-weight deflection of a simple beam are explicitly interrelated and are both dependent on the flexural rigidity, span and mass per unit length.

1.4.2.3 FP Innovation method for joisted timber floors

Hu (2007), and Hu and Chui (2004) proposed a criterion to control the vibration of light weight timber floors in Canada based on field investigations and surveys of building occupant opinions of how well floors performed. Different combinations of dynamic and static floor response parameters were considered as variables defining multi-factorial relationships between those variables and building occupant assessments of floor performance. It was concluded that all considered combinations could define a possible design criterion which were better than the already discussed displacement limit approaches (Section 1.4.2.1). Hu and Chui (2004) proposed that floors with $f_1/d_1^{0.44} > 18.7$ would have acceptable performance.

1.4.2.4 TRADA method

BS 6472 recommends using weighted a_{rms} between 0.34 and 0.45 m/s^2 , as a criterion for assessing floor motions under impact forces (BSI 1992). Research at the UK Timber research and Development Association (TRADA) applied that approach to joisted timber floors using a heel-drop impact as the disturbing force. Chui (1987) heel-drop field tests on six floors judged to have

acceptable, close to acceptable or unacceptable behavior. Based on the field measurements the following criteria in Fig. 1.5 were proposed:

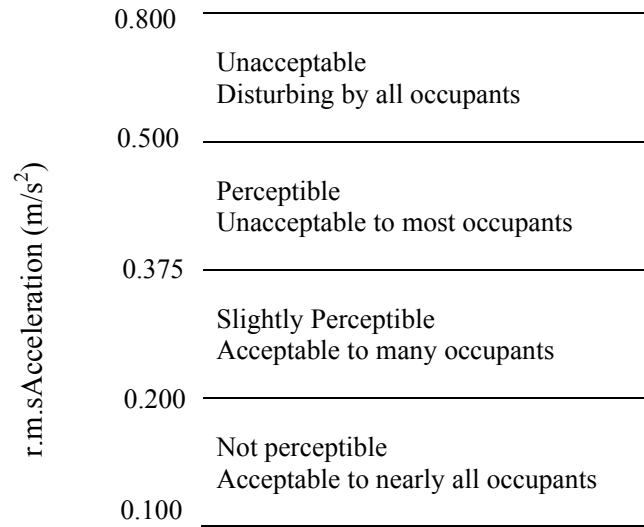


Fig. 1.5. Human perception versus weighted a_{rms} (Chui 1987)

Based on further investigations TRADA researchers suggested using weighted a_{rms} of 0.45 m/s² as the acceptance threshold for design of joisted timber floors (Smith and Chui 1988). Those researchers also suggested requiring $f_1 > 8$ Hz and using a defined heel-drop impact as the forcing function producing oscillatory floor motions. Fig. 1.6 shows actual and assumed design force functions, with the duration of the assumed force function (t_1) being 0.05-0.07 s and the amplitude of the assumed force function (P_0) being 0.7 the weight of person applying the heel-drop impact.

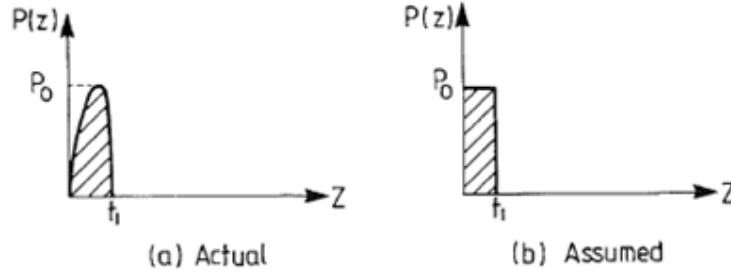


Fig. 1.6. Actual and assumed Shape of heel drop load (Smith and Chui, 1988)

Applying single degree of freedom (SDOF) analogy, an expression was developed for calculation of weighted a_{rms} caused by the assumed forcing function (Smith and Chui, 1988):

$$A_r = \frac{2000K_{\delta, w_n, t_1}}{m\pi f_0^2} \quad (1.3)$$

The detailed definition of K_{δ, w_n, t_1} can be found (Smith and Chui 1988). The TRADA became a preferred approach for vibration serviceability design of timber joisted floors because of its basis in engineering mechanics and practicality of application. The drawback is that the accuracy of the formula is limited to cases where the floor motion is dominated by the fundamental mode. Also the floor plan is assumed to be rectangular and supported along all edges. That in turn depends on construction details which could suppress clustering of low order modal frequencies. The Eurocode 5 method mentioned below (Subsection 1.4.2.5) has the advantage of being able to address cases where modal frequencies cluster, but is also limited to situations where plan shapes are rectangular and floors are simply supported along all edges.

1.4.2.5 Eurocode 5 method

Eurocode 5 (CEN 2004) provides three criteria for vibration serviceability design of rectangular joisted timber floors simply supported along all edges, based on research by Ohlsson (1982, 1988). The basis is consideration of low and high frequency components of human walking footfall impacts. Low frequency components generate semi-static vibration governed by the stiffness of a floor. High frequency components of impacts are addressed by considering the effects of a unit impulsive force (1 Ns) on the initial velocity of floor motions. A velocity response criterion accounting for the impulsive force is a function of a floor's modal response characteristics. A third criterion specifies a minimum f_1 requirement of 8 Hz. As already indicated (Subsection 1.4.2.4), a feature of the Ohlsson/Eurocode 5 approach is that it considers a case where low order modal frequencies cluster. This is a double edged sword because a consequence is design calculations are therefore quite complex even though a parameter termed n_{40} (equal to the number of first order modes with frequencies less than 40 Hz) is used as a surrogate for prediction of modal frequencies other than f_1 . Discussion by other experts has focused deeply on n_{40} for two reasons. Firstly, it has been found that the Eurocode 5 formula for calculation of n_{40} (equation 1.4) does not always lead to accurate results (Smith and Hu 1993, Weckendorf et al 2015). Secondly, the use of n_{40} assumes 40 Hz is the appropriate filtering frequency for removing contributions relatively high order modes make to floor motions (Weckendorf et al 2015). It is unclear how specific selection of 40 Hz is to the types of floors tested by Ohlsson (1982, 1988). Another aspect debated by experts is correctness of assuming the effective modal damping ratio is 1%, i.e. much less than values reported in the literature (Weckendorf 2015, Zhang et al 2013). Despite potential lack of generality and limitations, the Eurocode 5/Ohlsson approach is a potential blueprint for developing a widely applicable vibration serviceability design method(s) for lightweight floors.

$$n_{40} = \frac{B}{L} \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \frac{(EI)_L}{(EI)_B} \right\}^{1/4} \quad (1.4)$$

where L and B are floor length and width respectively, $(EI)_L$ is equivalent plate bending stiffness of the floor about an axis perpendicular to span direction and $(EI)_B$ is equivalent plate bending stiffness of the floor about an axis parallel to span direction.

1.4.2.6 Negreira et al method

Negreira et al (2015) examined previously suggested vibration serviceability design criteria for floors, plus the Maximum Transient Vibration Value (MTVV). The MTVV approach is based on ISO 2631-1 (ISO 1997). Those researchers proposed its use because they claim poor or no correlation exists between parameters such as f_1 , $d_{1,v}$ and a_{rms} and subjective rating assessments of floor performance. That claim is based on opinions they collected from sixty participants of different ages and both genders. That they would come to different conclusions to those reached by earlier investigators is no surprising because each group of researchers studied different limit combinations of samples from highly diverse floor and building occupant populations. The limitations of building occupant survey based criteria design methods has been discussed in detail by Weckendorf et al (2015). As they have stated, consistent with general statistical sampling and correlation study theory, such approaches can yield useful relationships if applied to tightly defined situations as done by Onysko et al (2000) for example. Such approaches can never establish a globally reliable relationship of parameters like d_1 and f_1 to how humans evaluate suitability of particular floors for particular building occupancy conditions. This is because there is no rigor underpinning researcher choices of study variables. It is important to bear in mind that human building occupant perceptions of floor performance are based on different sensory cues (e.g. visual,

audio), apart from floor motion cues that Negreira et al. have chosen to address. This illustrates amongst other things why laboratory and field based studies can result in different criteria for floors that are nominally the same. Claims by Negreira et al or others should be treated with caution because their basis is empirical at core.

1.4.2.7 Hamm et al criteria

Hamm et al (2010) carried out about 100 laboratory and field studies on timber floors with different construction details (e.g. timber beam floor, floors with heavy and light screed, timber concrete composite (TCC) floors and massive wood floors). Different methods such as heel-drop and sand bag drop impacts were used to excite floors and f_1 and damping ratios measured. Acceleration response of the floor from walking load as well as static displacement of the floor was measured. Subjective assessments of floor performances were collected. Limited correlation between measured f_1 and perceptions of floor performances was found. Hamm et al categorized floors as being “high demand”, “low demand” or “without demand” and defined f_1 and static displacement under 2 kN concentrated force point load (d_2) limits for each category. High demand floors ($f_1 > 8$ Hz, $d_2 < 0.5$ mm) are ones where vibrations are not annoying and motion would only be perceptible if someone concentrates on them. Low demand floors ($f_1 > 6$ Hz, $d_2 < 1.0$ mm) are ones for which vibrations are perceptible but not annoying. No demand floors ($f_1 = 4.5$ Hz, d_2 unlimited) are ones where motions are very perceptible and sometimes bother people. It helps highlight the dichotomy that characterizes literature reports of studies aimed at enabling subjective discrimination between floors having acceptable and unacceptable behaviors.

1.4.3 Vibration design of Low frequency floors ($f_1 < 8$ Hz)

1.4.3.1 Murray et al method (AISC 1997)

Low frequency floors often have relatively long spans and heavyweight (BSI 2008). Murray et al (1997) used a SDOF analogy to predict peak acceleration response of such floors walking impact forces. Table 1.2 constant force, damping ratios and acceleration limits suggested for various building occupancy situations. The importance of that table and the associated SDOF representation is that they demonstrate the possibility of addressing the need for differential performance levels for different building occupancies. It also highlights the lack of such emphasis in much of the research work done on high frequency floors.

Table 1.2. Constant force and acceleration limit (Murray et al., 1997)

	Constant Force (kN)	Damping ratio β	$\frac{a_0}{g} * 100\%$
Offices, Residents and Churches	0.29	0.02-0.05	0.5%
Shopping Malls	0.29	0.02	1.5%
Foot bridges- Indoor	0.41	0.01	1.5%
Foot bridges- Outdoor	0.41	0.01	5.0%

1.4.3.2 Hamm et.al. method

Hamm et al (2010) proposed an acceleration limit for low frequency floors additional to limiting for what they termed low demand ($f_1 = 6-8$ Hz, $d_2 < 1.0$ mm) no demand ($f_1 = 4.5$ Hz, d_2 unlimited) floors. The requirement is maximum acceleration due to walk impact forces should be $< 0.1 \text{ m/s}^2$. They recommend calculation of the maximum acceleration be calculated based on Murray et al (1997):

$$a\left(\frac{m}{s^2}\right) = \frac{F_{dyn}}{M^{mo} * 2\xi} = \frac{0.4 * F(t)}{m * 0.5 * (L * B) * 2\xi} \quad (1.5)$$

where M^{mo} is the modal mass of the floor, m is the mass area of the floor in (kg/m^2), B is the floor width in (m) ($B \leq 1.5 L$), L is the length of the floor in m , ξ is damping of the floor, $F(t)$ are effective harmonics of walking loads applied on the floor and can be read from Fig. 1.7. The factor 0.4 is introduced to account for forces not always being applied at the center of a floor.

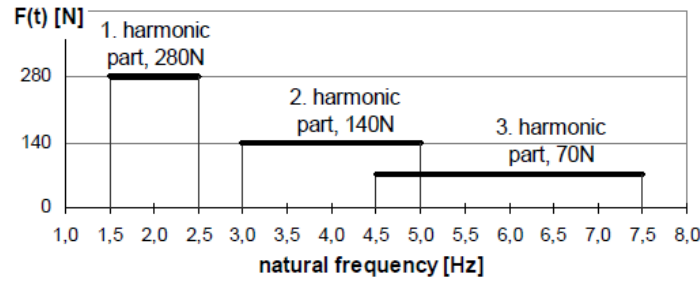


Fig. 1.7. $F(t)$ values suggested by Hamm et al (2010)

1.4.4 VDV method

So called Vibration Does Values (VDV) have been suggested as a method for evaluating floor motions defined as (ISO 10137, ISO 2631-1 and Setareh 2010):

$$VDV = \left[\int_{t=0}^{t=T} a_w^4(t) dt \right]^{1/4} \quad (1.6)$$

where T is the period which a person is exposed to vibration and $a_w(t)$ is the weighted acceleration.

Although VDV is sensitive to peak acceleration it estimates cumulative effects of a person's exposure to oscillatory motions. Employing frequency weighting makes it in essence an extended duration version of the approach embedded in the TRADA method for high frequency floors (Subsection 1.4.2.4). Recent research by (Hassan and Girhammer 2012) showed that VDV can be used to assess the vibration performance of timber floors. However, as mentioned by those authors there is no reliable VDV limit values adjusted for timber floors. More generally it highlights that choosing a criterion to base vibration serviceability design assessment method needs to go hand in hand with specification of a forcing function and response threshold levels appropriate to particular building occupancy situations. As with the MTVV (Subsection 1.4.2.6), the jury remains out on whether the VDV method has advantages over earlier methods.

1.5 Numerical models of light weight timber floors

Ohlsson (1982) used a grillage model to conduct dynamic analysis of timber floors, with beam element was used to model timber joists and cross beam elements used to represent floor sheathing. The approach predated ready availability of computers having sufficient memory to make FE models viable, and it should be viewed in that context. Filiatrault et al (1990) used a finite strip method to predict modal frequencies of lightweight joisted floors, neglecting shear deformations of joists. Again the context was desire to use an approach that was computational resource efficient. Smith et al (1993) developed a highly elegant and computationally efficient modal synthesis technique that accurately predicted mode shapes and modal frequencies of ribbed plates. It also could make time-history analyses and addressed effects of shear deformation, rotary inertia, discontinuity in the plate, semi-rigid attachment of the plate to ribs, and orthotropic material

characteristics. Once more the motivation was achieving computational efficiency. Special software was created for the purpose.

More recently attention has shifted toward use of FE methods, reflecting increased availability of cheap computing power and proliferation of general purpose commercial analysis software. Jiang et al (2004) developed a finite element model, based on the general purpose finite element software *VAST*, to predict the static and dynamic responses of joisted floors. Shell elements were used to model the floor sheathing, and beam elements represented joists and lateral (bridging) elements. Two node and four node connector elements were developed to connect the bridging to beams and sheathing to joists, respectively. Effects of shear deformation, rotary inertia and gaps between subfloor elements were considered in this model. The R^2 for correlation between measured and predicted d_1 was 0.8698, while the value for f_1 was 0.7327. Values of f_1 were overestimated, with the authors attributing the discrepancy to flexibility in floor support not mimicked by models. Alfoghaha et al. (1999) used the Abaqus commercial FE software to perform dynamic analyses of lightweight timber floors subjected to human walking forces to create curves interrelating f_1 , a_{rms} and mass of the floor. Close agreement was obtained between predicted and experimental results.

The above are just some of many examples of mostly FE numerical models that exist now for theoretical modal analyses, and a lesser extent time-history analyses of lightweight timber floors. Accuracy of predictions tends to relate directly to attention paid to representation of the construction details that characterize how floors are constructed.

1.6 Analytical joisted floor models

Chui (2002) modified the formulas proposed by Huffington and Hoppmann (1958) and Timoshenko and Woinowsky-Krieger (1959) to calculate the f_1 and static displacement d_p caused by a concentrated force P of rectangular plan joisted floors simply supported along all edges. Factors such as gaps between subfloor elements, partial composite action between subflooring and joists was accounted in the formulas based on McCutcheon (1986). Equations for d_p and f_1 are:

$$d_p = \frac{4p}{ab\pi^4} \sum_{m=1,3} \sum_{n=1,3} \frac{1}{\left(\frac{m}{a}\right)^4 D_x + 2\left(\frac{mn}{ab}\right)^2 D_{xy} + \left(\frac{n}{b}\right)^4 D_y} \quad (1.7)$$

$$f_1 = \frac{\pi}{2\sqrt{\rho}} \sqrt{D_x \left(\frac{1}{a}\right)^4 + 2D_{xy} \left(\frac{1}{ab}\right)^2 + D_y \left(\frac{1}{b}\right)^4} \quad (1.8)$$

where D_x , D_y and D_{xy} are longitudinal, transverse and torsion rigidity of the floor, a and b are length and width of the floor respectively, and ρ is mass per unit floor area. The flexural rigidities of the plate can be calculated as follows (Chui 2002):

$$D_x = \frac{EI_{cj}}{b_1} \quad (1.9)$$

$$D_y = \frac{\sum EI_b^i}{a} + \frac{EI_p b_1}{b_1 - t + \alpha^3 t} \quad (1.10)$$

$$D_{xy} = \frac{G_p h^3}{12} + \frac{C}{2b_1} \quad (1.11)$$

$$\alpha = \frac{h}{H} \quad (1.12)$$

$$EI_{cj} = EI_u + \overline{EA_1} h_1^2 - \overline{A} \overline{y}^2 \quad (1.13)$$

$$EI_u = EI_{joist} + b_1 EI_{s\perp} \quad (1.14)$$

$$\overline{EA_1} = \frac{b_1 EA_{s\perp}}{1 + 10 \frac{b_1 EA_{s\perp}}{S_1 L_1^2}} \quad (1.15)$$

$$\overline{A} = EA_{joist} + \overline{EA_1} \quad (1.16)$$

$$\overline{y} = \frac{h_1 \overline{EA_1}}{\overline{A}} \quad (1.17)$$

$$h_1 = \frac{d}{2} + \frac{h}{2} \quad (1.18)$$

$$\rho = m_j / b_1 + \rho_s h \quad (1.19)$$

where EI_{cj} is the composite EI of the joists; b_1 is spacing between joists; t is width of joists; a and b are span and width of the floor respectively; h is thickness of deck element; c is the width of one

beam; H is the total height of the floor system including beam depth and thickness of the slab; S_I is the load slip modulus of connections; L_I is the distance between sub-floor panel gaps perpendicular to span direction; EA_{joist} is the axial stiffness of joists; $EA_{s\perp}$ is unit subfloor axial stiffness in span direction; EI_{joist} is joist apparent EI ; $EI_{s\perp}$ is unit subfloor EI in span direction; m_j is mass per unit of the joists and ρ_s is density of sub-floor.

Accuracy of the formula was tested using results for twenty seven floors in which subflooring/sheathing was fastened to the joists and there were no bridging elements. It is suggested d_p can be calculated accurately using seven series terms in equation (1.7). Strong correlation was found between measured and predicted d_p . However, f_1 values were overestimated by equation (1.8). A more designer friendly formula, based on Rayleigh method, was proposed by Smith and Chui (1988) to calculate the f_1 of lightweight timber floors neglecting shear deformation and rotary inertia of joists:

$$f_1 = \frac{\pi}{2a^2} \sqrt{\frac{E_j I_j (n-1)}{(\rho_s h b + \rho_j c d (n-1))}} \quad (1.20)$$

where n is the number of joists. Adding the term $4w_0/a$ to the denominator represents the effect of a person of mass w_0 standing on a floor.

$$f_1 = \frac{\pi}{2a^2} \sqrt{\frac{E_j I_j (n-1)}{(\rho_s h b + \rho_j c d (n-1) + \frac{4w_0}{a})}} \quad (1.21)$$

1.7 Other Thesis Chapters

Chapter 2 consists of a manuscript titled “Vibration serviceability of glulam beam-and-deck floors”.

The chapter directly relates to objectives 1, 2, 3.

Chapter 3 consists of a manuscript titled “Finite element modeling and parametric study of glulam beam-and-deck floors”. This chapter directly relates to objectives 4 and 5.

Chapter 4 consists of a manuscript titled “Vibration characteristics of a large glulam beam-and-deck floor system”. This chapter directly relates to objectives 6 and 7.

Chapter 5 presents a general discussion that expands the topics addressed in Chapters 2, 3 and 4.

Chapter 6 provides conclusion and recommendations for future work.

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2 Chapter 2

VIBRATION SERVICEABILITY OF GLULAM BEAM-AND-DECK FLOORS

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(Submitted to Elsevier, Journal of Engineering Structures, Submission date 8/6/2017)

2.1 Abstract

Floor vibration serviceability problems can exist for floors constructed from low mass-to-stiffness ratio materials. This paper addresses low-amplitude vibration motion serviceability performance of glulam beam-and-deck floors. Such floors are a relatively new addition to the range of timber construction options for floors in non-residential buildings having spans up to about 10 m. As yet there exists no clear understanding if such floors exhibit dynamic motion characteristics similar to other timber floor systems, or if existing engineering vibration serviceability performance design concepts apply. Primary focus of discussion here is experimental investigation of how design variables such as span, support arrangement, beam spacing and addition of non-structural topping overlays alter vibration responses of beam-and-deck floors. Focus group subjective opinions of how laboratory-built floors responded to walking footfall impacts were collected as an indication of practicality of using engineering design decisions as controls on vibration serviceability performance of glulam beam-and-deck floors. Resulting test data is used to assess acceptability of simple engineering formulas as a means of estimating fundamental natural frequencies and displacements caused by a concentrated vertical static load. In general, results supported the notion

that appropriate choice of engineering design variables will effectively control the likelihood of vibration serviceability performance for situations involving motions created by human footfall impacts. No attempt is made to propose new vibration serviceability performances criteria or design methods. This is because it would conflict with ongoing international R&D efforts to create criteria and methods that apply across a wide range of lightweight floor construction methods. What is reported here is partial support of those efforts.

Keywords: Beam, deck, design, floors, glulam, lightweight, performance, timber, serviceability, vibration.

2.2 Introduction

Well established ways of constructing lightweight timber floors exist for various types of building designs and occupancy situations. Span capabilities differ widely, as do the controlling engineering design criteria. In some instances design is prescriptive, but that is confined to situations like construction of lumber joisted floors in houses and some other small buildings (Weckendorf et al 2015). Nearly always design of timber floors with spans greater than about 4 m is controlled by static or dynamic serviceability performance related criteria which are experience or structural mechanics based (e.g. Dolan et al 1999; Ohlsson (1988); Onysko et al. 2000; Weckendorf 2015). This reflects that all types of lightweight floors are prone to creating amplified motions that result in unacceptable levels of acceleration or velocity if not properly designed (Ohlsson 1982; Toratti and Talja 2006; Weckendorf et al 2015).

Starting about 40 years ago timber floors and buildings containing them began to depart substantially from traditional methods. For domestic dwellings and other relatively small buildings, this includes substitutions like replacing sawn timber joists with open-web joist products or wood I-

joists; substituting alternative deck products for sawn timber boards; and introduction of completely new construction methods (e.g. voided prefabricated panels). In some instances, this resulted in unanticipated vibration serviceability performance problems like low amplitude motions disturbing to humans which result from normal building occupant activities like walking. In reaction researchers in countries where timber construction is common began to study the phenomena. The focus of early research was mostly vibration performance of rectangular plan floors having relatively closely spaced joists (circa ≤ 400 mm), reflecting such systems were common, and remain so now (e.g. Ohlsson 1982; Onysko 1985; Polensek 1985; Smith and Chui 1988).

It was discovered that sources of problems often lie in construction product substitutions (e.g. use of low torsional rigidity joists, use of structural wood panels in lieu of floor boards) and alterations in how buildings are constructed. Some research studies have delved deeply into fundamentals of low-amplitude floor vibration motions and associated issues, but most studies have only sought practical remedies for minimizing chances of creating floors of given types having unacceptable in-service responses (e.g. Dolan et al. 1999; Hu et al. 2001). Design method suggestions often resulted from studies, but most are not ones founded on formal engineering methods. The indirect approach of defining acceptability of design solutions basing on surveys of building occupant opinions of suitability of floors was pioneered by Onysko (1985). As discussed in detail elsewhere, the essence of the Onysko and similar approaches is assuming building occupant opinions of floor performances are correlated with magnitudes of relatively easily estimated design parameters (e.g. fundamental natural frequency, displacement resulting from a concentrated gravity force) for particular combinations of floor construction method and building occupancy classification (Weckendorf et al 2015) . The ‘beauty’ of the approach is its simplicity of application, but it lacks of a sound scientific basis and unreliability if applied outside the strict limits of a

particular calibration exercise. Efforts have been made to create generally applicable vibration serviceability design criteria and methods based on scientifically sound dynamic principles (e.g. Ohlsson 1982; Smith and Chui 1988; Negreira et al 2015), but so far those also only apply to well-defined situations. Currently, the lack of generalized design criteria and methods (i.e. applicable to any type of lightweight floor system) presents a conundrum whenever a new type of construction system is created or an existing system is applied in a new context.

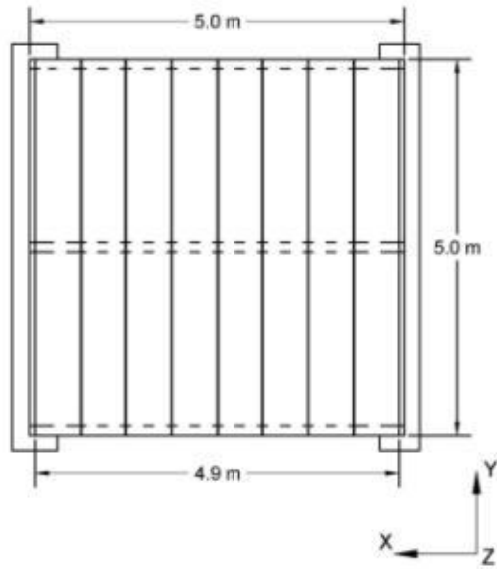
This paper addresses low-amplitude vibration motion serviceability performance of glued-laminated-timber (glulam) beam-and-deck floors. Such floors are a relatively new addition to the range of timber construction options for floors in non-residential buildings having spans up to about 10 m. Consequently, as yet there exists no clear understanding of whether such floors exhibit dynamic motion characteristics similar to other timber floor systems, or if existing engineering vibration serviceability performance design concepts apply. Primary focus of discussion is experimental investigation of how design variables such as span, support arrangement, beam spacing and addition of non-structural topping overlays alter vibration responses of beam-and-deck floors. Focus group subjective opinions of how laboratory-built floors responded to walking footfall impacts were collected as an indication of practicality of using engineering design decisions as controls on vibration serviceability performance of glulam beam-and-deck floors. Result and discussion sections use test data to assess acceptability of simply engineering formulas as a means of estimating fundamental natural frequencies and displacements caused by a concentrated vertical static load. No attempt is made to propose a new vibration serviceability performances criterion or a design method applicable to beam-and-deck floors. This is because it would conflict with ongoing international R&D efforts to replace a raft of *ad-hoc* approaches with criteria and methods that apply across a wide range of lightweight floor construction methods. Those international efforts

include activities supporting revision of the Canadian national timber design code CSA Standard O86 (CSA 2014) and Eurocode 5 (CEN 2004).

2.3 Test and Data Analysis Methods

2.3.1 Effects of construction variables

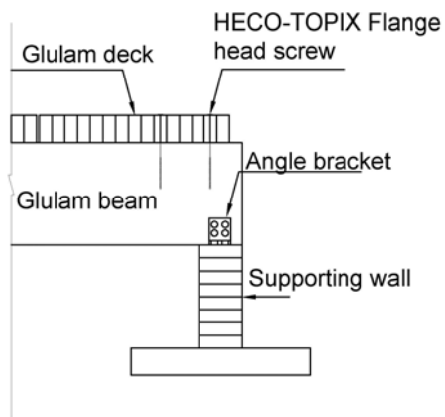
Nine floor configurations were tested representing effects of key construction variables on vibration responses of glulam beam-and-deck floors. Reference Floor 0 was designed to satisfy Ultimate Limiting States (ULS), but not Serviceability Limiting States (SLS) criterion applicable in Canada (CSA 2014). A further eight floors were constructed to address effects of decreasing beam span, increasing the number of glulam beams (i.e. decreasing the beam spacing), and adding intermediate beam support, and construction detail changes (e.g. adding non-structural topping materials). The general effect of modifications was to successively improve SLS performance. The adopted un-factored live load used was 2.4 kPa, which matches office floors and some commercial building occupancy classifications in Canada (NRC 2015). Fig. 2.1 shows the configuration for Floor 0 and Table 2.1 summarizes modifications creating Floors 1 to 8. Fig. 2.2 shows selected examples of modified configurations. Floor 1 and subsequent floors all met the SLS criterion that static deflection under an un-factored live load of 2.4 kPa did not exceed $\text{span}/360$ (WDM 2014).



(a)



(b)



(c)

Fig. 2.1. Reference floor configuration (Floor 0) (a) Plan arrangement (b) Constructed floor (c) Angle bracket connecting glulam beams to supporting wall

Table 2.1. Modification to Floor 0 creating Floors 1 to 8

Floor system	Arrangement / Modification made	Beams	
		Length (m) ^a	Number
0	Reference case, Fig. 1	5.0	3
1	Reducing beam length by 0.6m	4.4	3
2	Reducing beam length by 0.6m	3.8	3
3	Adding two beams, Fig. 2.2 (a)	3.8	5
4	Adding inclined screw to interconnect adjacent deck elements	3.8	5
5	Remove screws connecting adjacent deck elements, and adding middle support to create two equal beam spans of 1.9 m	3.8	5
6	Move support to one-third position to create two unequal beam spans of 2.5 m and 1.3 m, Fig. 2.2 (b)	3.8	5
7	Remove internal support, and add OSB sheathing	3.8	5
8	Pour non-structural concrete overlay on top of OSB	3.8	5

^aBeam length includes bearing distances of 130mm over each end support.

Floor 0 was constructed using three 130 mm x 304 mm x 5 m long beams, and eight 600 mm x 80 mm x 5 m long glulam deck elements, that were 20f-E Spruce-Lodgepole Pine-Jack Pine and No. 2 grade Spruce-Pine-Fir respectively (CSA 2014). Metal angle brackets and 6 mm diameter x 60 mm long proprietary HECO-TOPIX® Flange head screws were used to attach beams to short 130 mm thick support walls constructed from additional glulam beams. The support walls were themselves supported on a glulam deck elements fixed to a rigid reinforced concrete floor

(Figs. 2.1b and 2.1c). Glulam decks were attached to the beams using four 6 mm diameter x160 mm long HECO-TOPIX® Flange head screws per connection. Adopted support condition is similar to simple bearing supports commonly found in practice. Choice of HECO-TOPIX® screws was because their form and length permits reliable interconnection of deck element in a manner that prevents them from ‘bouncing’ on tops of beams during oscillatory system motions. The screws are partly threaded and plated with bright zinc. Substitution of similar proprietary screws should not alter floor response characteristics in a measurable way.

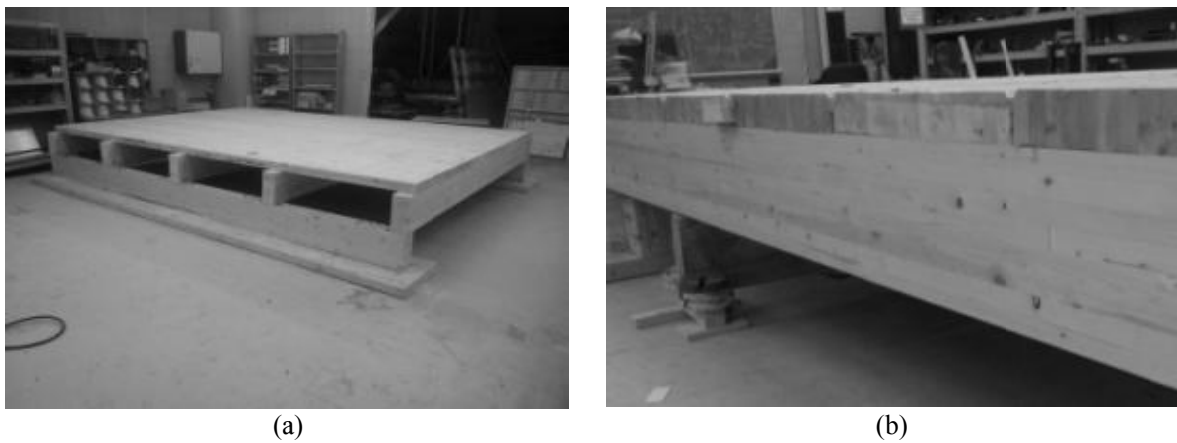


Fig. 2.2. Selected modified floor configurations (a) Floor 3: Floor with five 3.8m long beams(b) Floor 5: Floor with end and 1/3 span simple supports

In Floor 4 adjacent deck elements were interconnected using twenty pairs of 100 mm long HECO-TOPIX ®-CC screws having a shank diameter of 6.5 mm installed from opposing directions at an angle of 45° as shown in Fig. 2.3(a). Again form and length of the screws suited creation of secure connections between abutting parts of the system. The Oriented Strand-Board (commonly known as OSB) overlay in Floor 7 consisted of 1.2 m by 2.4 m sheets which were 15.5 mm thick, with 2R40/2F20 rating (CSA 2014). The strong OSB axis was oriented in across-beam direction

with sheets arranged so joints between them were staggered leaving 3 mm expansion gaps. OSB was fixed to glulam deck elements using 3.66 mm diameter by 65 mm long common wire nails. Nailing was at 150 mm intervals around sheet perimeters and 300 mm elsewhere, Fig. 2.3(b). That situation was investigated as a potentially simple way of enhancing composite action between glulam deck elements. The additional non-structural overlay added to create Floor 8 was 38 mm self-compacting concrete poured directly onto the OSB layer of Floor 7. The concrete had a compressive strength of 30 MPa, density of 2300 kg/m^3 , maximum aggregate size of 10 mm, having a slump of 500 mm. The type of high workability concrete is typical of that used in practice, and was poured by an experienced crew.

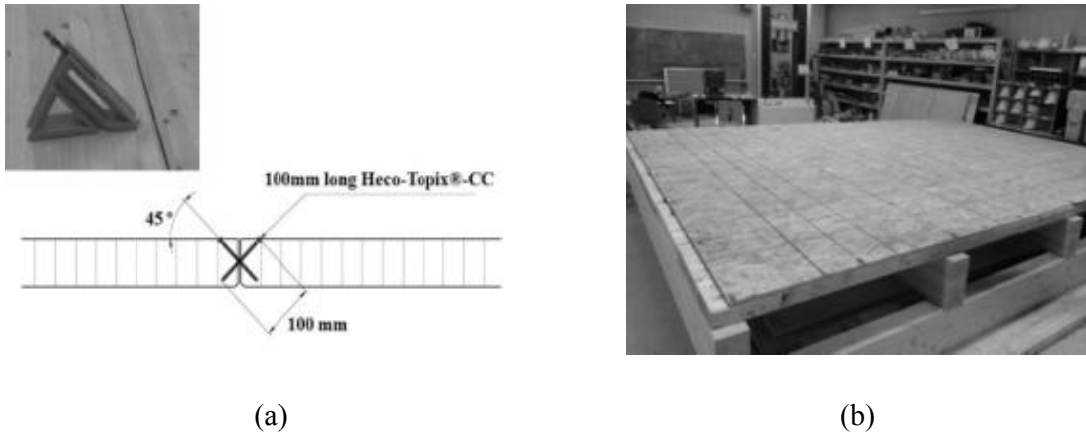


Fig. 2.3. Examples of special construction details (a) Floor 4: Deck elements interconnected by inclined screws (b) Floor 7: Non-structural OSB added

Prior to construction, Forced Vibration Tests (FVT) were conducted to determine the average flexural rigidities (i.e. values of modulus of elasticity in bending $\times$ second moment of area) of glulam beams and deck elements (Ewins 2000). Modulus of elasticity values reported below are

flexural rigidities divided by second moments of area determined using measured element cross-section dimensions. Ambient Vibration Tests (AVT) were conducted to measure modal frequencies and mode shapes of each test floor (Brincker and Ventura 2015). Also, static deflection caused by a 1 kN vertical point load (d_1) was measured at the center-span position of each floor, using a dial gage with 0.01 mm precision referenced from the laboratory floor.

In FVT a beam or deck element was suspended using two flexible rubber ropes at nodal points which were located at 0.22 and 0.78 of the length (Fig. 2.4). This approach achieves free-free end conditions and was taken because it results in the ‘purest’ estimates of element dynamic elastic modulus (Smith and Chui 1992). Other support methods add contaminated influences into definition of the span(s) and damping characteristics. Beams/deck elements were dynamically excited using an instrumented hammer, with acceleration responses measured using a piezoelectric accelerometer. The excitation point was kept constant and the accelerometer moved along the specimen during modal testing. Beam/deck element surfaces were cleaned and sanded prior to mounting the accelerometer. Double sided tape was used to attach the accelerometer to the surface of beam or deck elements. Locations of the excitation and accelerometers measurement were selected to give the best mode shapes and frequencies based on trial testing. Recorded time domain data was transformed to frequency domain data by Fast Fourier Transform (FFT) technique, using RT Pro 6.33 software (Brüel & Kjær, 2008). The fundamental natural frequency of a specimen was obtained from the resulting Frequency Response Functions (FRF) to estimate longitudinal modulus of elasticity, E , using equation (2.1), (Smith and Chui 1992; Haines et al. 1996):

$$E_x = 0.946 \frac{f_0^2 l^4 \rho}{d^2} \quad (2.1)$$

where f_0 is the fundamental natural frequency, l is the length of the specimen, d is depth of the specimen and ρ is the mass density measured by direct weighing the specimen. Average values derived were $E_{x-beam} = 10.8$ GPa and $E_{x-deck} = 10.0$ GPa, based on three tests per specimen. The extracted material properties are used in predictive formulas discussed in subsequent parts of this paper.



Fig. 2.4. Suspended beam element for FVT

In AVT a Tromino data acquisition system was used to measure the dynamic response of test floors using grids of measuring points, as illustrated in Fig. 2.5. Selected measuring grids were ones proving sufficient motion observation points for fully reliable detection of low amplitude responses of modes having frequencies in the range ≤ 60 Hz (Ohlsson 1988; Cantieni and Biro 2005; Smith et al. 2009). Specifically the objective was to define mode shapes in the frequency range ≤ 40 Hz, which is the range considered by the Eurocode 5 design practice for timber joisted floors (CEN 2004), based on investigations by Ohlsson (1982, 1988). Tests on each floor were

replicated multiple times until it was clear that extracted modal response information was correct and reproducible, with one Tromino sensor kept in a constant position as the a master response node for synchronized observations made with roving sensors, Fig 2.5. The time duration for each set of measurements was 120 seconds, during which a trolley was moved across the floor following a motion pattern that excited all relevant modes. The trolley had three wheels and a mass (10 kg), that was insignificant compared to masses floors themselves. It was moved using a long handle avoiding need for the operator to stand on the floor and influence responses. Collected data was preprocessed and analyzed using the Enhanced Frequency Domain Decomposition (EFDD) technique as implemented by the Artemis Extractor 5.3 software tool (Structural Vibration Solutions A/S, 2011). EFDD is an advanced technique which overcomes limitations of normal FDD (Jacobsen et al. 2006). In EFDD the autocorrelation function is used to extract modal frequencies and damping ratios. The single degree of freedom (SDOF) spectral density function (SDOF bell) is identified around a resonance peak and transformed back to the time domain using an Inverse Fast Fourier Transform (IFFT). The associated natural/modal frequency and equivalent viscous damping are obtained from the estimated zero crossing time and logarithmic decrement of the corresponding autocorrelation function (Brincker and Ventura 2015).

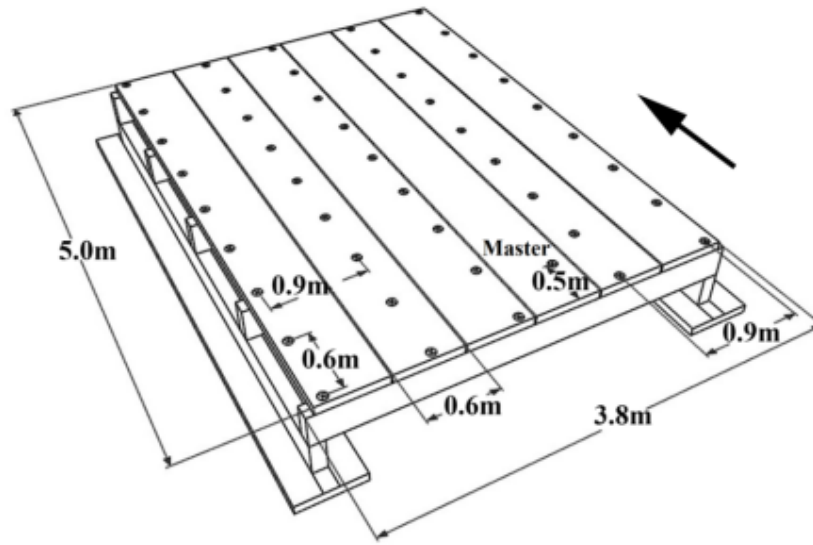


Fig. 2.5. Measurement points on a typical floor and direction of roving sensor movements

2.3.2 Focus group opinions of floor performances

Focus group subjective opinions of how laboratory-built floors responded to walking footfall impacts were collected as an indication of practicality of using engineering design decisions as controls on vibration serviceability performance of glulam beam-and-deck floors. Deliberately, there was no attempt to use collected data for additional purposes like creation of an ad-hoc empirical design rule applicable to vibration serviceability performance assessment of glulam beam-and-deck floors. This was, because as already discussed in the Introduction, and elsewhere (e.g. Weckendorf et al. 2015), such practices have no generality and thus have limited utility. Nevertheless, the authors employed methods based on those of researchers who did seek to create design methods applicable to specific construction methods and building occupancy situations (Hu 2013).

The subjects whose opinions of floor performance characteristics were canvassed consisted of groups of 20 university students and staff. The mean age of the subjects was 35 years and their mean weight 70 kg, with 75% being men. A criterion for selecting particular people was creation of groups of which half lived in buildings having reinforced concrete or similarly relatively rigid floors, and the other half lived in buildings with relatively flexible timber floors. The objective of this was to avoid bias in the basis from which each test floor would be judged. Tests were conducted in silence in an enclosed laboratory without windows, depriving subjects from extraneous visual and auditory cues. No interruptions occurred while opinions were being formed. First subjects were asked to walk freely on a floor. Second they sat on a 520 mm height four leg stool placed about 300 mm from the center of the floor to see if they felt any movement or bounce when a male researcher weighing 78 kg walked on the floor in a set pattern at the normal pacing rate. The walking pattern involved the researcher walking twice in both the span and width plan directions of the floor, then diagonally from one corner to another. For consistency he always wore the same type of shoes. At the end of tests subjects were asked to fill out a questionnaire. Questions established their demographic characteristics (e.g. age, gender and type of floor in their residence) and opinions about intensity of vibrations they had experienced and acceptability of the test floor's performance, Table 2.2.

Questionnaire responses were used to create individual discrete rating classifications using the scale adopted by Hu (2013): 1 - Definitely unacceptable, 2 - Unacceptable, 3 - Marginal, 4 - Acceptable, and 5 - Definitely acceptable. Because rating levels are discrete values, rating level statistics (i.e. mean values and standard deviations) are purely indicative of how floors were rated by subjects as a group.

Table 2.2. Questions and rating scale used in subjective assessment

Question	Rating scale answers
- I felt the floor move or bounce while I am moving on the floor.	1 - Strongly agree
- This is not a satisfactory floor for office or similar buildings.	2 - Agree
- I felt the floor move or bounce while the researcher moved on the floor.	3- Somewhat agree
- I was annoyed by the floor movement bounce.	4 - Disagree
	5- Strongly disagree

2.4 Results and discussion

2.4.1 Effects of construction variables

Table 2.3 summarizes mid-span deflections (d_1), the three lowest order modal frequencies (f_i , $i = 1, 2, 3$), and the damping ratios for the fundamental mode (ζ_1). The three lowest mode shapes always exhibit one half-wave deformation in the floor length (X axis) direction. Mode shapes 1 and 3 were symmetric while mode shape 2 was anti-symmetric in the floor width direction (Y axis). Fig. 2.6 shows illustrative mode shapes for Floor 3.

Table 2.3. Modal frequencies, damping ratio and d_1 for floors

Floor			f_1 (Hz)	f_2 (Hz)	f_3 (Hz)	ζ_1 (%)	d_1 (mm)
No.	L (m)	Features					
0	5.0	3 beams	11.1	13.7	21.8	1.3	0.58
1	4.4	Ditto	13.2	18.1	23.6	1.1	0.43
2	3.8	Ditto	16.7	22.9	30.7	1.0	0.32
3	3.8	5 beams	21.6	24.1	30.8	2.2	0.21
4	3.8	Ditto, joined deck ^a	22.7	26.1	30.9	2.3	0.20
5	3.8	Ditto, span/2 support	31.1	34.0	42.7	2.8	N/A
6	3.8	Ditto, span/3 support	29.8	34.1	41.5	2.9	N/A
7	3.8	Ditto, OSB	20.2	21.1	29.3	1.5	0.17
8	3.8	Ditto, OSB + conc.	18.8	20.8	29.8	3.1	0.07

^a Deck elements interconnected using inclined screws.

As would be expected reducing span, while keeping the number of beams constant (Floors 0 to 2), decreased d_1 . However, the reduction was less than the cubic dependence applicable to a simply supported beam according to simple beam theory. Increasing the number of beams from three to five (Floor 3 vs. Floor 2) but keeping the span constant decreased d_1 by 34%, i.e. much less than the if associated stiffness coefficients were linearly proportional to the number of beams. Interconnecting deck elements using inclined screws reduced d_1 by only 0.01 mm (Floor 4 vs. Floor 3). This practically negligible effect was because the connections were not sufficiently rigid to make the deck elements behave as a plate. However, leaving out such screws could for instance result in differential movements that crack non-structural concrete toppings and ceramic tiles.

Adding topping layers of OSB (Floor 7) and OSB and poured concrete (Floor 8) reduced d_1 by 19 and 67% respectively, relative to the corresponding displacement of Floor 3. This supports previous observations that toppings can greatly increase two-way bending action of decks and ribbed plates (Forabaschi and Vanin 2015; Onysko et al. 2000). A consequential conundrum is whether designers can effectively apply vibration serviceability design criteria (especially those strongly dependent on estimating d_1) when they typically will not be deciding what non-structural elements will be installed in buildings.

In essence findings illustrate importance of the two-way action of beam-and-deck floors. Findings for effects of construction variable on natural frequencies also show this, and the potential pitfall of assuming effects of construction modifications are fathomable without detailed test and/or numerical investigation.

Effects of altering floor span on modal frequencies are shown directly by results for Floors 0 to 2, and indirectly by Floors 5 and 6. For the simple span floors (Floors 0 to 2) the three lowest modal frequencies increased when the span was reduced, because those are all first order modes (m = number of half-wave in the span direction = 1). In the cases of Floors 5 and 6, both of which had an intermediate beam support, f_1 was less for the latter because it had a larger effective span due to the offset position of the intermediate support. Conversely, f_2 and f_3 values for Floors 5 and 6 were about the same. This reflects bi-directional mode shaping has influences on modal stiffness and mass characteristics that may not be intuitive.

Mode	3D View	Top View	Side View
Mode 1			

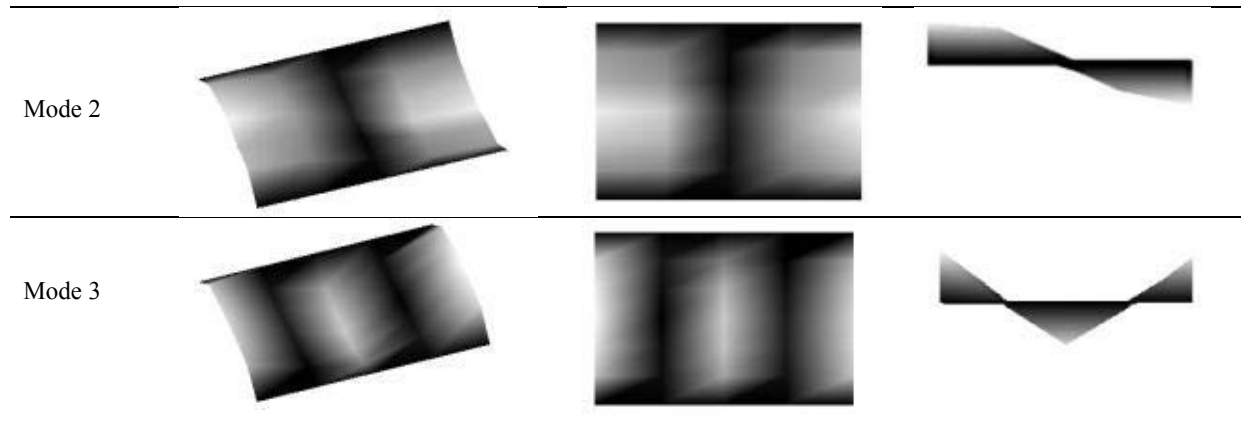


Fig. 2.6. First order mode shape of a typical glulam beam-and-deck floor (Floor 3)

Modal frequency separations ($f_2 - f_1$ or $f_3 - f_2$) followed different trends for the various floors. For example, modal frequency separation ($f_2 - f_1$) for Floor 0 was less than those for Floors 1 and 2. In practical terms there is an important implication of the result for the effect of floor span. Amplifications of floor motions resulting from effects of modal clustering are most likely for systems like Floor 0 where the span is similar to or greater than the width of a floor. Sensitivity of low order modal frequencies to addition of two extra beams (Floor 3 vs. Floor 2), which approximately halved beam spacing, resulted in 29% increase in f_1 , rather than 67% that would apply if fundamental natural frequencies were only dependent on the number of beams. It is therefore clearly not reasonable to oversimplify notions of how altering construction details will affect fundamental natural frequencies. For modal frequencies f_2 and f_3 effects of adding beams to a system are even less intuitively deducible. However, in general it is valid to think influences of deck element stiffness on modal frequencies increase with the order of the mode shape considered.

Any measure aimed at supplementing continuity of decking or deck stiffening that does not also substantially increase modal masses may be expected to increase low order natural frequencies

of floors (e.g. Floor 4 or 7 vs. Floor 3). However, as results in Table 2.3 illustrate, this did not happen. Interconnecting deck elements using inclined screws increased modal stiffnesses slightly more than the associated modal masses, but the reverse occurred when OSB topping was added. When both OSB and poured concrete toppings were present there were more pronounced reductions in modal frequencies (e.g. f_1 for Floor 8 was 18.8 Hz vs. 20.2 Hz for Floor 7) than when there was only OSB. This was because any gains in modal stiffnesses were less than gains in modal masses.

Material damping for timber and derived products like glulam is less than 1% of the critical damping ratio (Weckendorf et al. 2015). Therefore system damping present in vibrating floors made of such material is only greater than material damping when frictional or inertial damping are also present. This reflects that lightly loaded or unloaded glulam beam-and-deck systems should not exhibit intra-system impact damping, because deck elements are effectively anchored to the beams. Frictional damping occurs when parts slide or otherwise move relative to each other at contact surfaces between elements, between elements and the support system or is derived from especially when $L = 3.8$ m as floor profile is not particularly shallow. Below the fundamental modal damping ratio ζ_1 is taken to be a surrogate for system damping.

An interesting finding is ζ_1 decreased quite significantly as floor span was reduced (Floors 0 to 2). This reflects dependency of damping on system compliance, which reduces when the span is reduced. The suspected mechanistic reason being less friction damping occurs at contact surfaces between beams and deck elements when span is reduced. Conversely, ζ_1 increases due to the presence of intermediate beam supports (Floors 5 and 6) because of friction damping at contact surfaces between beams and supports. Comparing results for Floors 2 and 3, increasing the number of beams from three to five more than doubled ζ_1 , which is also attributed to increase frictional

damping. Interconnecting adjacent deck using inclined screws had small effect on damping (Floor 4 vs. Floor 3). Here the reason is that only a small quantity of screws were involved. For all floors without non-structural overlays ζ_1 ranged from 1.0 to 2.9%, with the value roughly equal to material damping plus supplementary damping proportional to the number of contact surfaces between beams and supports and beams and deck elements.

Floor 7 that had an OSB topping had a ζ_1 of 1.5%, while Floor 8 which had layers of OSB and poured concrete topping had a ζ_1 of 3.1%. The approximate doubling of system damping is attributed solely to presence of the poured topping layer substantially increasing friction and inertial damping, due to the mass of the concrete being much greater than that of the OSB layer.

2.4.2 Prediction of f_1 and d_1 using simple formulas

Various simply closed form formulas have been proposed for prediction of f_1 and d_1 . Intent is those formulas be used in applications of simple design criteria like empirical methods mentioned in the Introduction. Table 2.4 illustrates capabilities of typical formulas based on f_1 and d_1 values predicted by equations (2.2) and (2.3) (Torrati and Talja 2006). Equation (2.3) matches that presented by Mohr (1999), with the authors having confirmed (formulated in appendix A) it also can be derived analytically from orthotropic plate theory of Timoshenko and Woinowsky (1959).

$$f_1 = \frac{\pi}{2\sqrt{m}} \sqrt{D_x \left(\frac{1}{L}\right)^4} \quad (2.2)$$

$$d_1 = \frac{0.024L^2}{D_x^{3/4} D_y^{1/4}} \quad (2.3)$$

where: D_x and D_y are longitudinal and transversal flexural rigidities of the floor, respectively, L is the floor span, and m is the mass per unit floor area. It is assumed $D_x = (EI)_{cbeam}/b$ and $D_y = E_{x-deck} t^3 b / (12(b-t+\phi^3 t))$. ($(EI)_{cbeam}$ = composite EI of beam including shear (Hu 2007), ϕ is the ratio of deck element thickness to floor height, b = beam spacing, t = deck element thickness). The value of shear modulus was considered as 0.7 GPa (Xiao 2014).

Table 2.4 comparisons suggests that in cases where the deck elements are not acting as a continuous plate (Floors 1 to 4) equations (2.2) and (2.3) predict both f_1 and d_1 with accuracy that might well be acceptable to design engineers. This is logical because those are circumstances where assumptions underpinning the formulas are most valid. In other situation, like when floors have non-structural overlays f_1 is estimated with lesser accuracy, particularly for Floor 8 which has both OSB and poured concrete toppings. The formula estimate of d_1 for the floor with OSB overlay only (Floor 7) was moderately inaccurate, but for Floor 8 the predicted d_1 was grossly inaccurate. No results are reported for Floors 5 and 6 because the formulas do not apply in those instances. Overall the comparisons demonstrate need for judicious caution.

Table 2.4. Comparison between measured and simple formula predictions of f_1 and d_1

Floor	0	1	2	3	4	7	8
	f_1 (Hz)						
Measured (Hz)	11.1	13.2	16.7	21.6	22.7	20.2	18.8
Equation (2.2)	10.5	13.4	17.9	23.2	23.6	22.4	14.7
	d_1 (mm)						
Measured	0.58	0.43	0.32	0.21	0.20	0.17	0.07
Equation (2.3)	0.60	0.47	0.36	0.21	0.21	0.21	0.17

2.4.3 Focus group opinions of floor performances

Fig. 2.7 summarizes how various floors in the test program were rated by focus group participants. Floors 0 and 1 had ratings of 1.0, meaning they were unambiguously judged unacceptable according to the FPInnovation classification categories (Hu 2013). Shortening floor span to create Floor 2 increased the average rating value to 1.2, but that is also an unacceptable classification. Adding two extra beams to create Floor 3 increased ratings considerably, with the average value rising to 3.1 (i.e. marginally acceptable). Vibration performance ratings improved further for Floors 4 to 8. Results for Floors 3, 7 and 8 imply that beyond a certain point increasing the transverse stiffness of floors (e.g. adding non-structural toppings) has diminishing returns in terms of how humans will rate floor performance. Reasons include interactions, and in some instances counteractions, of effects construction modifications have on modal masses, stiffnesses and damping. After a one-third span intermediate support (Floor 6) was introduced the average rating value rose to 4.1. Introduction of a mid-span intermediate support (Floor 5) resulted in an average rating of 4.3. Net effect of such intermediate support was reduction of the effective spans of beams leading to lower motion amplitudes. According to the FPInnovation subjective rating system Floors 4 to 8 had acceptable performances, based on average ratings. However, none of the studied floors were judged to be definitely acceptable.

Focus group opinions demonstrate humans can sense effects different construction modifications have on vibration serviceability performances of glulam beam-and-deck floor systems, individually and collectively. The caveat to this is assessment conditions were artificial and it is not certain the same capability would exist under “natural” circumstances. It would be invalid to suppose the results prove or disprove existence of reliable correlations between floor

response characteristics that engineers can predict (e.g. d_1 , f_1) and in-service acceptability of specific floor design solutions. Proponents have hypothesized such correlations exist (e.g. Hu 2013, Hu et al. 2001, Onysko 1985) and possibly empirically based criteria are an acceptable basis for design of some floor systems. The so far illusive challenge for proponents is how to frame limits on the reliable application of such approaches.

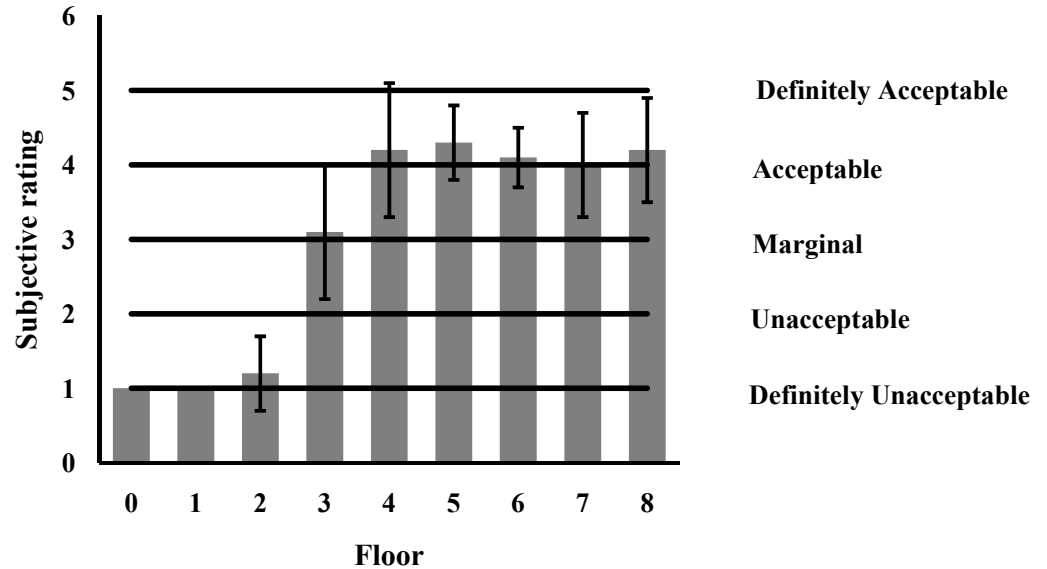


Fig. 2.7. FPIInnovation vibration performance ratings of test floors (Data are presented as average values $\pm$ one standard deviation)

Table 2.5 illustrates application of the simplest possible types of empirical design criteria, i.e. ones that limit maximum displacement resulting from a defined loads (d_1), or fundamental natural frequency (f_1). In that table the adopted limits on d_1 and f_1 are taken from Canadian and US proposals intended to apply to rectangular joisted timber floors. Comparison of outcomes obtained using the criteria shows the differing approaches would result in inconsistent design decisions (i.e.

whether performance is acceptable, marginal or unacceptable). Also to note is that neither criterion mimics assessment given by the focus groups in all cases. Specific limitations $d_1 < 8/\text{span}^{1.3}$ (mm) and $f_1 > 14$ Hz are undoubtedly not optimal for beam-and-deck floors. However, use of different values would not have eradicated inconsistencies between criteria and between them and focus group assessments. Using alternative proposed criteria based on both d_1 and f_1 , or them and other variables does not lead to a different conclusion. This reflects that increasing complexity of fitted surface functions means they fit more calibration data points but does not necessarily improve their interpolative or extrapolative capabilities. The authors hold to opinion that use of any such approaches are stopgaps pending definition of structural dynamics approaches that match predictions of motions to internationally agreed limits for motions humans can tolerate.

Table 2.5. Illustration of dichotomy of ratings of test floors obtained applying proposed vibration serviceability design criteria*

Floor Criteria	0	1	2	3	4	5
$d_1 < 8/\text{span}^{1.3}$ (Onysko 1985)	P	P	P	P	P	P
$f_1 > 14$ Hz (Dolan et al. 1999)	P	P	P	P	P	P
Focus group rating	F	F	F	M	P	P

*F: Fail (Unacceptable), P: Pass (Acceptable), M: Marginal

2.5 Conclusions

The experimental investigations reported here confirms the anticipatable expectation that altering construction details of glulam beam-and-deck floors alters static deflection and modal characteristics of such systems. However, not all the influences of construction variables were anticipatable. Simple engineering design formulas were able to estimate observed static displacements caused by a concentrated gravity force and fundamental natural frequencies to reasonable accuracy for some floors, but not others. This implies need to use relatively complex static and dynamic structural analyses models. Focus group evaluations of acceptability of low amplitude oscillatory motions of laboratory-built floors demonstrated individually and collectively humans could sense effects each investigated construction variation had. There were major inconsistencies in how focus group evaluated floor performances versus predictions based on proposed design criteria applicable to other types of timber floors. What is reported here will contribute toward ongoing international efforts to create vibration serviceability design criteria and methods that apply across a wide range of lightweight floor construction methods.

2.6 Acknowledgements

The authors acknowledge financial support from the Canadian Natural Science and Engineering Research Council (NSERC). They also acknowledge the company Western Archrib that provided glulam test materials.

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3 Chapter 3

FINITE ELEMENTMODELLING AND PARAMETRIC STUDY OF GLULAM BEAM-AND-DECK FLOORS

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(Ebadi, M. M., Doudak, G., & Smith, I. (2017). Finite-Element Modeling and Parametric Study of Glulam Beam-and-Deck Floors. *Journal of Structural Engineering*, 143(9), 04017106.)

3.1 Abstract

Small-amplitude cyclic vertical motions of timber floors perceived as unacceptable by humans are commonly the result of walking impact forces. Contemporary vibration serviceability guidelines mainly require prediction of static displacement and modal frequencies of floors. This study used an advanced finite-element (FE) analysis approach to model glulam beam-and-deck floor systems. This permits prediction of static displacement and modal response characteristics that closely match values determined by testing full-scale floor. The verified modeling method is used to show how variations in floor details such as span and floor width affect the vibration behaviours of these floors. This shows that changing the floor width has little effect on the fundamental frequency and mid span deflection of a floor, but higher-order modes are strongly affected. The broad conclusion is that reliable prediction of parameters engineers used to predict vibration serviceability of such floors depends on use of appropriate models. Appropriate models are ones that incorporate

deep system effects on motions stemming from the layered nature of beam-and-deck element floors and depths of glulam elements used as the beams.

Keywords: beam-and-deck floors, design, dynamic response, natural frequencies, serviceability, static displacement, vibration performance

3.2 Introduction

Complexities present in timber structures that include orthotropic material properties, semi-rigid connection between parts, and complicated geometry and support conditions indicates the analysis of their substructures and complete superstructures is often beyond the capabilities of traditional engineering approaches. Floors are often amongst the most complicated substructures in such buildings and lend themselves to application of finite-element (FE) and other numerical methods of estimating deformations and force flows caused by static or dynamic forces. This paper addresses prediction of small-amplitude elastic static displacements and modal frequencies of glued-laminated-timber (commonly abbreviated to glulam) beam-and-deck floors. To date, such systems are most common as roof and floor substructures of post-and-beam super structures of non-residential buildings. Such roofs and floors have widely spaced parallel beams to which striplike deck layers arranged in the across-beams direction are mechanically attached (Fig. 3.1). Sheathing is typically nailed on top of plank decking systems as to resist lateral forces acting on the diaphragm. Because of their low ratios of modal mass to stiffness, timber floors can be prone to vibration serviceability performance problems, and beam-and-deck floors are unlikely to be an exception (Weckendorf et al. 2015).

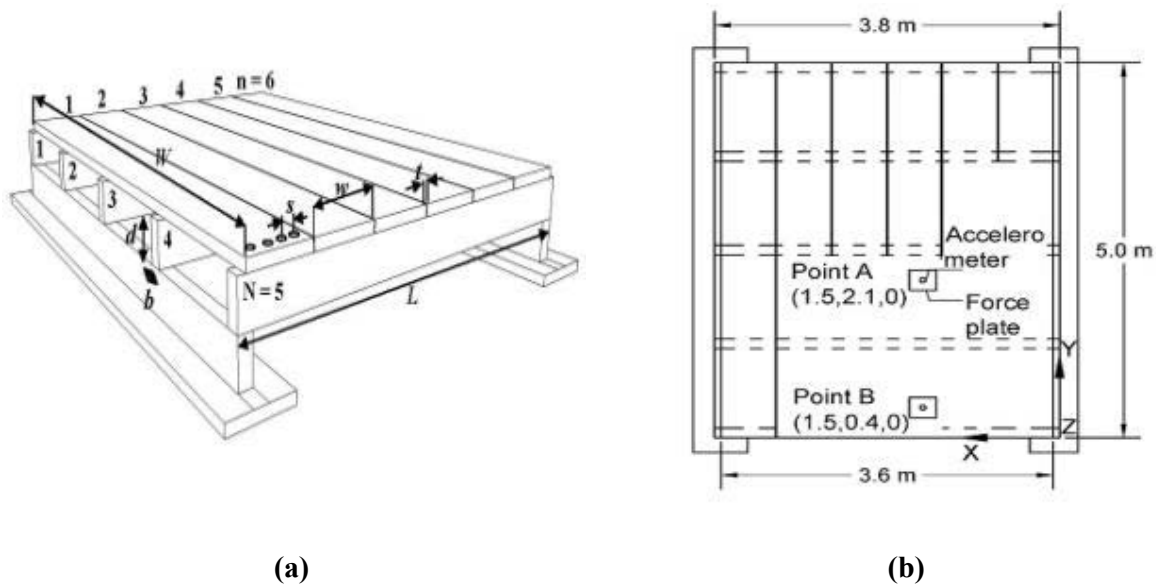


Fig. 3.1. Floor 4. a) Constructed floor b) Plan arrangement with location of applied accelerometer and force plate

Although various authors have elucidated concern that vibration serviceability is a concern engineers should address, there is no reliable quantification of the extent of problems. Survey information reported by Hu and Chui (2004) indicates approximately 1 in 10 lumber-joisted lightweight floors constructed according to prescriptive requirements for houses and other small buildings have performances that building occupants judge as unacceptable. Although instances of unsatisfactory performance are likely to be quite rare for engineered systems like glulam beam-and-deck element floors, it would be unwise to ignore that possibility.

Various investigations have studied vibration serviceability performances of timber floors. Ohlsson (1982) used grillage models to represent floors with closely spaced joists having semi-rigidly attached wood-based panel product subfloor sheathing. His model considered the effect of torsional stiffness but radial inertia was neglected for all members. In another study, Filiatrault et al.

(1990) used finite-strip method to present similar floor; however, the effect of shear deformation was neglected. They conducted a sensitivity analysis to investigate the effect of each floor design parameter on the frequency responses of floors. Smith et al. (1993) developed a model based on the free interface modal synthesis method, which could predict complete system modal frequencies and masses from those of the joists and other components. The component modal characteristics could be obtained through experiments or subsidiary analyses and account for various deformation characteristics they possessed. Jiang et al. (2004) developed a finite element model by using the general-purpose commercial software *VAST*, to predict the static and dynamic response of floors with closely spaced parallel joists. Shell element were used to model the sub-floor sheathing, and beam elements were used to model the joists and lateral reinforcement members. Two-node and four-node connector elements were developed to connect the bridging to the beams and the sheathing to joists, respectively. The effect of shear deformation, rotary inertia and gap between sub-floor layers were considered in the models by Smith et al. (1993) and Jiang et al. (2004). Alfoghaha et al. (1999) used the commercial software Abaqus (1996) to conduct dynamic analysis of light weight timber floors subjected to human walking forces. In total, the various methods have demonstrated the practicality of using numerical models to predict static and dynamic responses of lightweight timber floors. The adopted approaches reflected parallel evolutions in analytical methods and increasing availability of computer-based software tools.

As recently summarized elsewhere (Weckendorf et al. 2015), a range of vibration serviceability design criteria of differing complexities have been proposed for different types of lightweight timber floor systems. Most of those criteria are empirically based, and none apply directly to beam-and-deck floors. However, there is a general adoption by suggested criteria of static deflection caused by a force of 1 kN, usually denoted d_1 , and the fundamental natural

frequency, usually denoted f_1 , as floor response parameters related to discriminative assessment of the likely serviceability performances of floors. The pan-European model timber design code Eurocode 5 (CEN 2014) also uses n_{40} , which is the number of first order modal frequencies less than 40 Hz, as part of the recommended design criteria for floors with closely spaced parallel joists. Consequently the presumption underpinning analysis here is that engineers require models at least capable of predicting d_1 , f_1 and parameters like n_{40} (i.e. can predict relatively low-order modal frequencies). The remainder of this paper addresses development, verification and use of FE models for glulam beam-and-deck systems.

3.3 FE Models

3.3.1 Concepts and methods

The models discussed in this paper were created by using the commercial software Abaqus 6.12, (Called subsequently as Abaqus). Type B32 three-node quadratic Timoshenko orthotropic beam elements were used to model glulam beams because they incorporate effects of transverse shear deformation and rotary inertia. That was considered important because the physical arrangement of beam-and-deck floors indicates that they will not always behave as shallow substructures. Such elements have six degrees of freedom at each node, which can be used to model slender and stout beams. The deck layers and non-structural overlays (e.g. Oriented Strand-Board sheathing, or OSB) were modeled by using orthotropic shell elements because those have much smaller thickness than the depth of the beams. Those were type S4R four-node, doubly-curved reduced integration with hourglass control shell elements. Discontinuities that exist between deck layer strips and sheets of OSB or other overlay materials were included in the models by

appropriately meshing the FE arrangement (i.e. deck and overlay layers geometries of FE models replicated physical gap geometries).

Connections made with fasteners between deck layers and beams are semirigid and therefore only develop partial composite action between layers in floor systems. Where construction parts (e.g. deck and beam) are interconnected in physical floor systems by using mechanical fasteners such as self-tapping screws, it was replicated by appropriately interconnecting FE model nodes using spring elements. The horizontal stiffnesses of each spring element was considered equal to elastic slip characteristics of fasteners subjected to lateral load in direction parallel to the beam and parallel to the deck layer span. This allowed interface relative slip between construction parts. The same approach applied for non-structural overlays semi rigidly attached to deck layers. In all floors, the beams were modeled as pin end supported with lateral movements restrained. Finite-element models had two vertical, very rigid, zero-mass link elements at each fastener location. The length of an upper rigid link was half the deck thickness, and the length of a lower rigid link was half of the beam depth, which prevented vertical separation or overlap of deformed construction layers.

Three different analyses including static, frequency and modal dynamics were undertaken in this model. The general static analysis with predefined Newton method and automatic stabilization was applied to find the mid span deflection of the floor under 1-kN point load. Frequency analysis was performed to extract mode shapes and natural frequencies. The default Lanczos eigenvalue solution method in ABAQUS was used because it could extract the modal characteristics for the range of frequencies of interest, which were those up to approximately 50 Hz based on the analysis filtering frequency of 40 Hz Eurocode 5 employs for other types of lightweight timber floor systems (CEN, 2014). The modal dynamic analysis with zero initial condition was used to obtain the time

history acceleration response of the floor under the time-varying load obtained from the ball drop test. This analysis technique is a cost-effective modeling method compared with other dynamic analysis techniques (e.g explicit direct integration), which can be used for dynamic finite-element analysis of linear systems. The automatic meshing was applied in the *ABAQUS* preprocessor after the seed points were assigned to the model. Mesh sensitivity analyses revealed that solutions converged rapidly as grid spacing was reduced. Typically modal frequency predictions converged for mesh a spacing of 0.1 m or less. Reported modal frequencies correspond to 0.025 m.

For the purposes of engineering analysis, glulam beams can be regarded as an orthotropic material with elastic constants defined by parallel to length, width, and depth axes of material symmetry 1, 2, 3 respectively; i.e. elastic moduli E_1 , E_2 , E_3 shear moduli G_{12} , G_{13} , G_{23} and Poison's ratios ν_{12} , ν_{13} , ν_{23} . Initial stiffnesses responses of fasteners interconnecting beam and deck elements, or other elements can be regarded as linear elastic and defined by the initial tangent stiffness, k . Lateral load-slip responses of each connector was measured via push-out tests performed according to ASTM D1761 (ASTM 2012) (Appendix B). The values of primary material constants required by FE models and geometrical properties of various floors are defined in Tables 3.1 and 3.2, respectively. These values were compiled from results found by Ebadi et al. (2016), Xiao (2014) and Wood Hand Book (FPL 2010).

Table 3.1. Primary elastic constants and density for FE models*

Constant	Value (units)	Source of value
<i>Glulam beams</i>		
E_1	10.8 GPa	Ebadi et al. (2016)
E_2	1.1 GPa	FPL (2010)
E_3	0.7 GPa	FPL (2010)
G_{12}	0.7 GPa	Xiao (2014)
G_{13}	0.7 GPa	Xiao (2014)

G_{23}	0.1 GPa	FPL (2010)
ν_{12}	0.3	FPL (2010)
ν_{13}	0.3	FPL (2010)
ν_{23}	0.3	FPL (2010)
Density	470 kg/m ³	Ebadi et al. (2016)
Glulam deck elements		
E_1	10.0 GPa	Ebadi et al. (2016)
E_2	1.0 GPa	FPL (2010)
E_3	0.7 GPa	FPL (2010)
G_{12}	0.7 GPa	Xiao (2014)
G_{13}	0.7 GPa	Xiao (2014)
G_{23}	0.1 GPa	FPL (2010)
ν_{12}	0.3	FPL (2010)
ν_{13}	0.3	FPL (2010)
ν_{23}	0.3	FPL (2010)
Density	489 kg/m ³	Ebadi et al. (2016)
Mechanical fastener in connections:		
$k_{\text{deck-to-beam}}$	540 kN/m	Ebadi et al., (2016)
$k_{\text{OSB overlay-to-deck}}$	300 kN/m	Hu (2007)

* Glulam beams and decks were Spruce-Pine-Fir (SPF) with stress grade of 20f-E and NO. 2, respectively (CSA 2014).

Table 3.2. Geometrical properties of various floors

Floor	L	W	N	d	b	t	w	s
1	5	5	3	0.304	0.13	0.08	0.6	0.2
2	4.4	5	3	0.304	0.13	0.08	0.6	0.2
3	3.8	5	3	0.304	0.13	0.08	0.6	0.2
4	3.8	5	5	0.304	0.13	0.08	0.6	0.2
5 ^a	3.8	5	5	0.304	0.13	0.08	0.6	0.2
6 ^b	3.8	5	5	0.304	0.13	0.08	0.6	0.2

Note: L = beam length, W = floor width, N = number of beams, d = beam depth, b = beam width, n = number of deck parts, t = deck thickness, w = width of deck parts, s = spacing of deck-beam screws.

^a As floor 4 but with inclined screw interconnecting adjacent deck parts.

^b As floor 4 but with non-structural OSB layer nominally nailed to deck parts.

3.3.2 FE model verification studies

A comprehensive series of dynamic tests was undertaken by the authors at the University of Ottawa (Ebadi et al, "Vibration serviceability of glulam beam-and-deck floors" Submitted to

Engineering structures). The results of those tests are used in this study to verify the aforementioned FE modeling techniques. This encompasses predicting static deflections (d_1), low-order mode shapes, low-order modal frequencies, and time-history responses caused by dropping a 5-kg medicine ball onto floor surfaces.

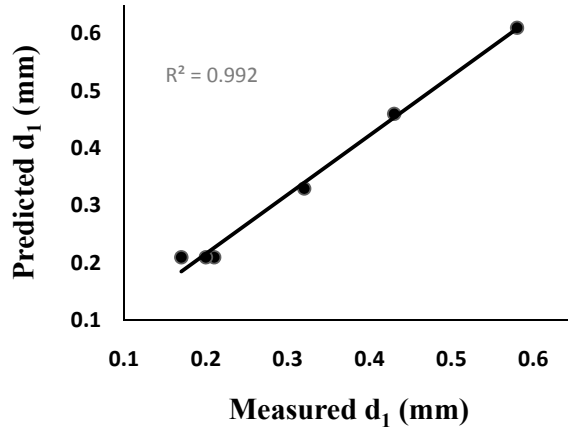
Table 3.3 compares predicted and experimentally derived estimates of d_1 and low-order modal frequencies f_i , ($i = 1, 3$) for a range of floors with simply supported (SS) beams with different spans, different floor widths, and in one case, with deck element interconnected at abutting edges using inclined self-tapping screws and, in another case, with a non-structural OSB overlay added.

Table 3.3. Comparison of FE model and test derived d_1 and f_i for floors with SS beams

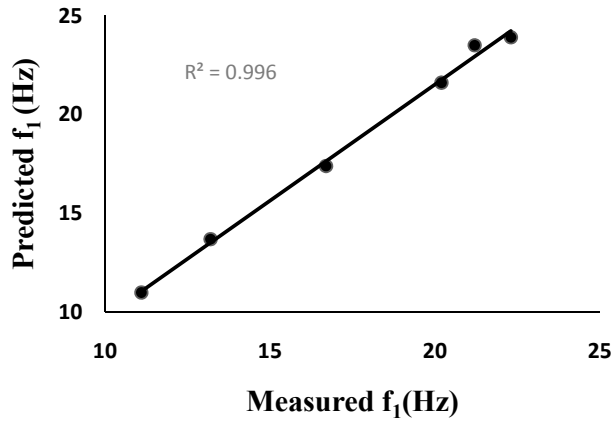
Floor	d_1 (mm)			f_1 (Hz)			f_2 (Hz)			f_3 (Hz)		
	test	model	% diff	test	model	% diff	test	model	% diff	test	model	% diff
1	0.58	0.61	5.2	11.1	11.0	-0.9	13.7	14.6	6.6	21.8	21.5	-1.4
2	0.43	0.46	6.9	13.2	13.7	3.8	18.1	17.7	-2.2	23.6	25.7	8.9
3	0.32	0.33	3.1	16.7	17.4	4.2	22.9	21.2	-7.4	30.7	31.9	3.9
4	0.21	0.21	0	21.6	23.5	8.8	24.1	26.8	11.2	30.8	32.5	5.5
5	0.20	0.21	5	22.7	23.9	5.3	26.1	30.7	17.6	30.9	32.6	5.5
6	0.17	0.21	23.5	20.2	21.6	6.9	21.1	24.8	17.5	29.3	30.3	3.4

Fig. 3.2 summarizes the overall levels of agreement between d_1 and fundamental natural frequencies (f_1 values) for floors in the test series, and Fig. 3.3 shows a typical comparison between test-derived and FE-model predicted mode shapes. In all investigated situations, the models

correctly identified the sequence of mode shapes that were identified during experiments, including higher-order modes not shown in this paper. Overall, it was found that the model predicted characteristics of SS beam-and-deck floors accurately. Discrepancies between test and model results are attributed to variability in material properties and friction between parts not included in the models.



(a)



(b)

Fig. 3.2. Test versus predicted d_1 (a) and f_1 (b) of tested SS floors (Different points corresponds to different floors)

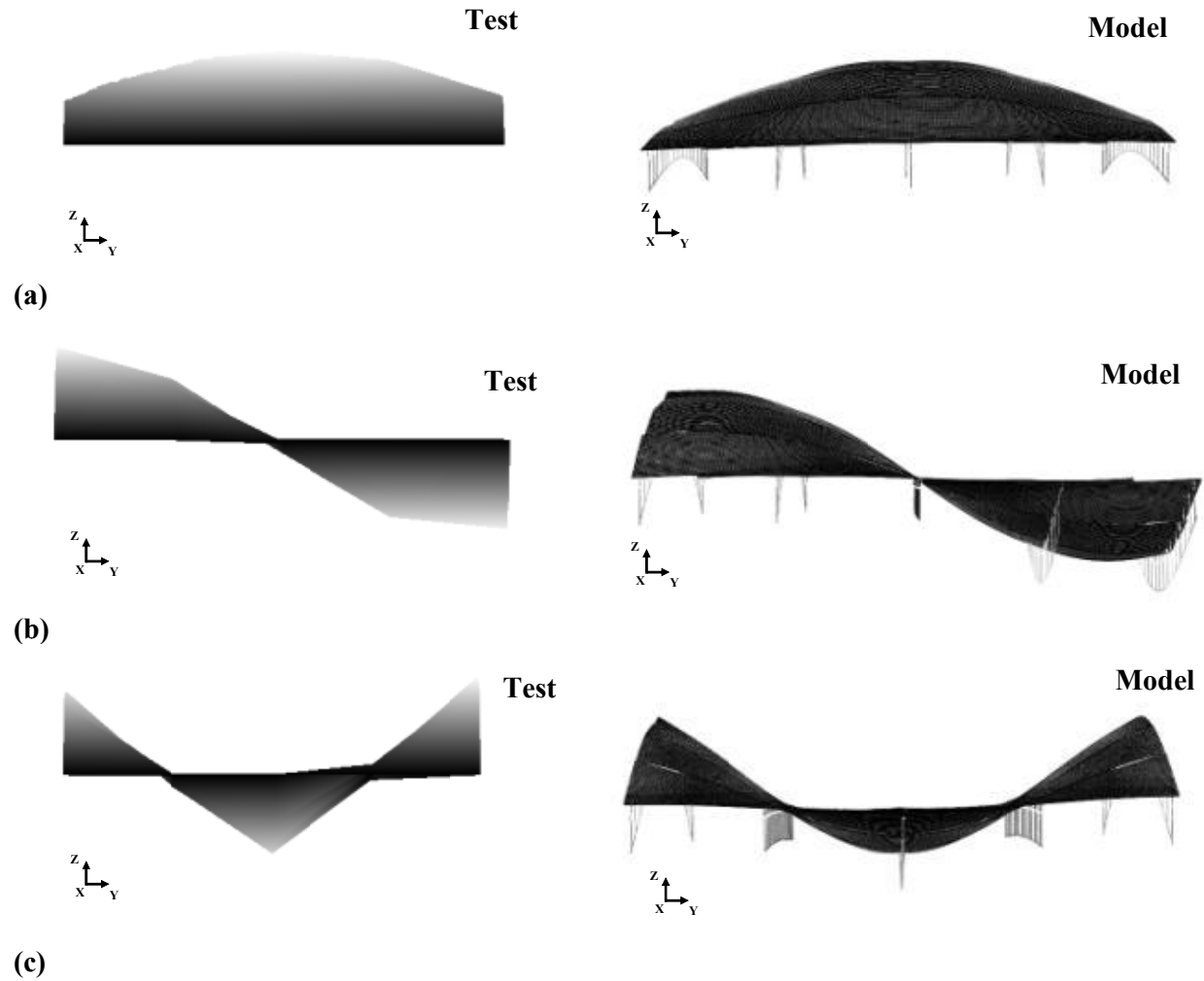


Fig. 3.3. Typical test and FE model mode shapes: Floor 1

Fig. 3.4 shows a typical time-history responses for a 5-kg medicine ball dropped from a height of 200 mm onto a force plate fixed to Floor 4, rebounding prevented by catching the ball on its first bounce. The location of force plate and accelerometer is shown in Fig. 3.1 (b). Such an impact was chosen because of the repeatability of the approach and ability to measure the dynamic excitation force history precisely (Kim and Jeon 2014). The time history of measured force obtained

from force plate was applied in the model. Modal dynamic analysis was carried out using first 3 extracted modes from frequency analysis and damping ratio of 2.2% which was obtained from measurement for first modal frequency of the floor. As the figure illustrates, the model predicted time-history acceleration responses of floors well before $t = 0.15$ s. The shift in amplitude and phase after $t = 0.15$ s might be related to the effect of filters on the measured signal. Applied filters removed noise, including a high pass filter to eliminate the effect of measured DC offset. Consequently, predictions of the first positive peak acceleration were very accurate, as Table 3.4 illustrates. This capability of the FE models is important because a number of proposed vibration serviceability design criteria are based on peak velocity, or peak or root-mean-square acceleration responses to defined impulsive or impact forces (Weckendorf et al. 2015).

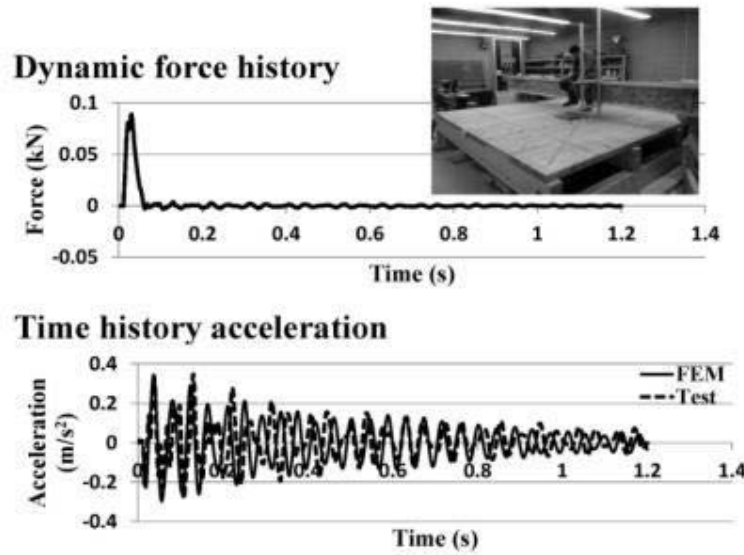


Fig. 3.4. Typical time-history response to a medicine ball impact: Floor 4 (Relative to point A)

Table 3.4. Comparison test and FE model estimates of peak acceleration (a_{peak})

System	$a_{\text{peak}}(\text{m/s}^2)$	
	test	model
Floor 4*	0.34	0.34
Floor 4**	0.51	0.51
Floor 6*	0.24	0.23
Floor 6**	0.41	0.36

*Point A **Point B

3.4 Effects of Design Variable on Floor Responses

In this section the validated FE modelling techniques are used to demonstrate how alteration of various parameters influences static deflection and modal responses of glulam beam-and-deck systems. Emphasis is placed on d_1 and low-order modal frequencies (f_i values) because those are the parameters mostly used by engineers to assess suitability of a potential design solution controlled by vibration serviceability criteria. Except where specifically mentioned, illustrative analyses apply to systems with 130-mm-wide by 304 –mm-deep glulam beams and 600-mm-wide by 80-mm-thick glulam deck parts having material properties in Table 3.1.

3.4.1 Effect of floor aspect ratio

Table 3.5 illustrates the effect doubling floor width (i.e. halving floor plan aspect ratio L/W) but keeping beam spacing the same has on d_1 and f_i . As the tabulated values illustrate, if floors are relatively wide (W/L greater than unity), the value of d_1 is insensitive to changes in the width of a

floor if the concentrated force of 1 kN is placed at the mid floor position. For floors that are narrow, that would not be the case but tend to apply to footbridges rather than building floors. By contrast, there is a very marked effect of altering W/L on f_i for $i > 1$, with very distinct clustering of low-order modal frequencies of the illustrative 10.0-m-wide floor.

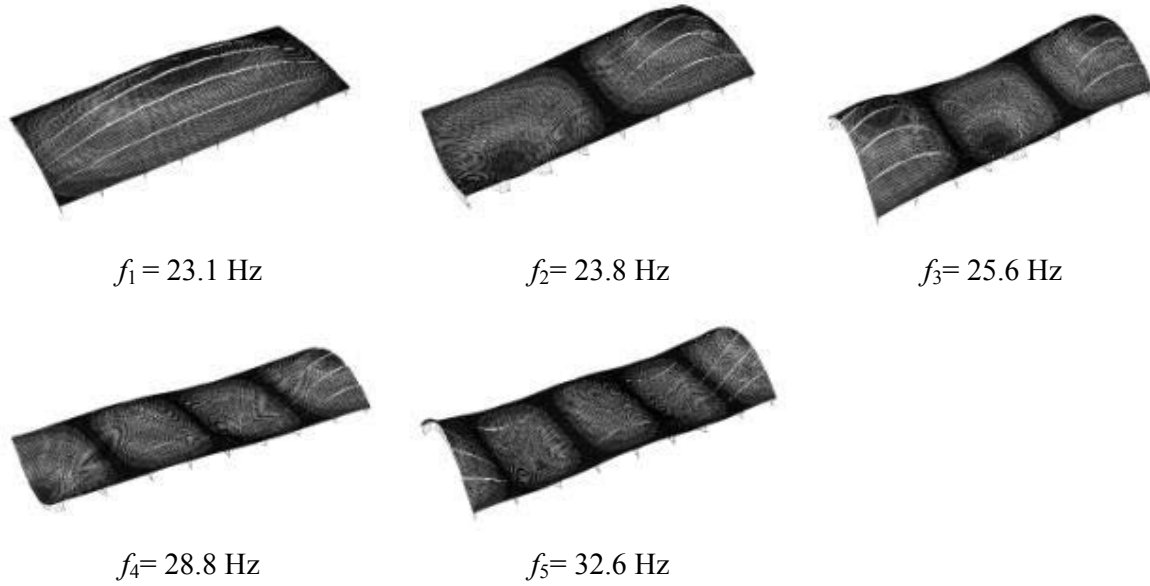
Table 3.5. Effect of doubling floor width on d_1 and f_i ($i = 1, 5$)*

<i>Properties</i>	d_1 (mm)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)	f_4 (Hz)	f_5 (Hz)
Width $W = 5$ m, number of beams $N = 5$						
Prediction	0.21	23.5	26.8	32.5	43.5	65.7
Width $W = 10$ m, number of beams $N = 9$						
Prediction	0.21	23.1	23.8	25.6	28.8	32.6
%	0	-1.7	-11.2	-21.2	-33.8	-50.4

Note: * $L = 3.8$ m, $d = 0.304$ m, $b = 0.13$ m, $n = 6$, $t = 0.08$ m, $w = 0.6$ m, $s = 0.2$ m. (A plus sign indicates an increase and a minus sign indicates a decrease)

Fig. 3.5 shows the first five mode shapes for the wider floor, illustrating the reason. All those modes are so-called first-order ones, indicating there is half-sine wave curvature in the parallel to beams direction in each case, making the flexural behaviours of the beams the dominant influence on the associated modal frequencies. Interactions of floor span and width with other variables such as thickness of the deck layer (t) define mode shapes in the across-beams direction. However, it is valid to think that within the practical range of choices of beam-and-deck system arrangements decreasing the ratio L/W will usually increase proneness to modal clustering, which in turn increases proneness to large-amplitude motions caused by footfalls or other impacts on floor surfaces. The tendency has been observed for same lightweight floors with closely spaced parallel

joists (Ohlsson 1988; Smith and Chui 1988). This implies the need to consider modal frequencies other than the fundamental value when designing beam-and-deck floors for vibration serviceability.



* $L = 3.8 \text{ m}$, $W = 10.0 \text{ m}$, $N = 9$, $d = 0.304 \text{ m}$, $b = 0.13 \text{ m}$, $n = 6$, $t = 0.08 \text{ m}$, $w = 0.6 \text{ m}$, $s = 0.2 \text{ m}$.

Fig. 3.5. Low-order mode shapes obtained for a floor with 9 beams*

3.4.2 Effect of beam depth

Because static deflection and low-order modal frequencies are typically highly dominated by flexural behaviours of the beams, altering beam depth has a strong influence on floor system performance characteristics. Table 3.6 shows how altering beam depth effects d_1 and f_i ($i = 1, 3$) for a SS floor with three 5-m-long beams and L/W of 1.0. The chosen beam depths are ones commercially available in North America, ranging from 304 mm to 570 mm. The chosen example illustrates that because all of the static deflected shape and modal stiffnesses and masses depend on

characteristics of beam and deck layers as controlled by the system arrangement, there are no direct proportional relationships of d_1 and f_i values to the I -ratio. This demonstrates that calculation of the system level stiffness coefficient associated with d_1 or modal stiffnesses and masses associated with a particular f_i value are the only reliable ways of predicting the performances of particular beam-and-deck systems.

Table 3.6. Effect of beam depth on d_1 and f_i ($i = 1, 3$)*

Beam depth (mm)	d_1 (mm)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)
<i>(I-ratio)**</i>				
304(1.00)	0.61	11.0	14.6	21.5
418(2.60)	0.28	15.9	19.7	27.9
570(6.59)	0.13	21.9	23.8	37.6

Note: * $L = 5.0\text{m}$, $W = 5.0\text{m}$, $N = 3$, $b = 0.13\text{m}$, $n = 8$, $t = 0.08\text{m}$, $w = 0.6\text{m}$, $s = 0.2\text{m}$.

** I -ratio is the ratio of beam second moments of area indexed to that of a 304 mm deep beam.

3.4.3 Effect of intermediate support and boundary conditions

Changing the effective spans of beams by introducing intermediate supports or encasing their ends can be an alternative effective way of improving floor performance characteristics. For example, experiments by the authors showed that introducing a line wall support to an initially simply supported 3.8-m-long floor with five beams and L/W of 0.76 increased the three lowest modal frequencies by 44, 41 and 39 % respectively (Ebadi et al, "Vibration serviceability of glulam beam-and-deck floors" Submitted to Engineering Structures, University of Ottawa, ON). Similarly,

illustrating effectiveness of altering the beam span FE model predictions for a 5-m-long floor with three beams and L/W of 1.0, encasting beam ends reduced d_1 by 64 %, and increased the three lowest modal frequencies by 85, 58 and 76 % respectively (Table 3.7). As the two given examples demonstrate, there is no reliable rule of thumb for the reduced static deflection and high-end frequency tuning benefits that will accrue from reducing effective spans of beams. Nevertheless, it is correct to conclude that it is a very effective design strategy for improving vibration serviceability performance of beam-and-deck floors. A caveat to this is that encasting ends of glulam or timber beams can be technically challenging, and caution is required to avoid vibration and sound transmissions serviceability problems with continuous span floors (Smith, 2003).

Table 3.7. Effect of beam support conditions on d_1 and f_i ($i = 1, 3$)*

Support conditions	d_1 (mm)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)
SS^a	0.61	11.0	14.6	21.5
FF^b	0.22	20.3	23.1	37.9
%	-63.9	+84.5	+58.2	+76.3

Note: $L = 5.0\text{m}$, $W = 5.0\text{m}$, $N = 3$, $b = 0.13\text{m}$, $n = 8$, $t = 0.06\text{m}$, $w = 0.6\text{m}$, $s = 0.2\text{m}$; Plus sign indicates an increase; minus sign indicates a decrease.

^a Floor with simply supported beams

^b Floor with fixed-fixed supported beams

3.4.4 Effects of deck layer thickness

As has already been mentioned, the features that influence the effective flexural rigidities of a system in parallel to span and across beams can strongly influence tendencies of particular systems to exhibit clustering of low-order modal frequencies. Table 3.8 illustrates the typical effects that altering deck layer thickness has on d_1 and f_i ($i = 1, 3$). The effects on d_1 of increasing deck thickness were strong for the two systems considered (i.e. having the same floor width but different

numbers of beams and spans). Proportional reductions in d_1 for $t > 80$ mm were similar, but that is coincidental, given that t was not the only variation between the systems. The results of f_i demonstrate clearly the complexity of the issues involved in assessing effects of altering design variables of dynamic responses of beam-and-deck floors. For both systems analyzed, all modal frequencies reduced with increases in deck thickness, reflecting that, because the decks consisted of strips of glulam with a finite width of 600 mm ($w = 0.6$ m), modal masses increased more than modal stiffnesses. Fundamental modal frequencies therefore reduced, but because modal stiffness to mass ratios altered differently for different modes, increasing deck thickness increased frequency separations between modes. Such effects are a double-edged sword in terms of vibration serviceability performance-related design decisions. Lowering of modal frequencies increases the risk of resonating human body organs (Weckendorf et al. 2015), but reduction clustering of modal frequencies decreases the likelihood of amplification of motions when floors are subjected to impact forces.

Because the effects of variables are not independent, it is not possible to generalize about economic impacts of decisions such as choice of deck element thickness. However, it is reasonable to expect that tradeoffs that result in simultaneous increases or reductions in both deck thickness and spacing of beams should not be expected to alter the total volume of material greatly.

Table 3.8. Effect of deck thickness on d_1 and f_i ($i = 1, 3$)

Deck thickness (mm)	d_1 (mm)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)
$L = 4.4\text{m}, W = 5.0\text{m}, N = 3, d = 0.304\text{m}, b = 0.13\text{m}, n = 7, t = 0.08\text{m}, w = 0.6\text{m}, s = 0.2\text{m}$				
80	0.46	13.7	17.7	25.7
130	0.33	12.1	16.8	30.1
175	0.26	10.9	16.1	35.1
$L = 3.8\text{m}, W = 5.0\text{m}, N = 5, d = 0.304\text{m}, b = 0.13\text{m}, n = 6, t = 0.08\text{m}, w = 0.6\text{m}, s = 0.2\text{m}$				
80	0.21	23.6	26.8	32.5
130	0.15	20.1	24.1	34.1
175	0.12	18.1	22.4	38.9

3.4.5 Effect of cross beam direction gaps

Table 3.9 demonstrates how changing the amount of continuity in a deck by altering the number of across-beams gaps between glulam deck parts (i.e. widths deck parts) affects d_1 and f_i ($i = 1, 3$). The gaps were each 8-mm wide, indicating that adjacent deck parts would come in contact with each another. However, as they were attached to beams by using self-tapping screws, that would act compositely with beams over their widths. As the results indicate, the number of gaps had relatively small influences on d_1 and f_i values. The explanation lies in the low modulus of elasticity of the glulam deck in the beam span direction (E_2), and to lesser extent in the relatively nominal extent of the fasteners attaching deck to beam parts ($s = 200$ mm).

Not discussed in detail in this paper are the effects of altering design variables like the elastic modulus of deck layer parts in their across width direction (E_2), beam widths (b), and spacing of fasteners inter connecting deck and beam layers (s). This is not to imply such factors cannot be used to significantly modify and control the static and the dynamic responses of beam-and-deck floors. They are not specifically emphasized in this paper because they are less likely than discussed design variable to be economically viable ways of significantly altering floor responses.

For example, for Floor 4, changing the number of fasteners per deck element to beam connection from 4 to 13 altered predictions of low order modal frequencies by a maximum of 1.3 %.

Table 3.9. Effect of number gaps in deck layer on d_1 and f_i ($i = 1, 3$)*

Number of gaps	d_1 (mm)	f_1 (Hz)	f_2 (Hz)	f_3 (Hz)
7	0.61	11.03	14.60	21.50
3	0.59	11.09	14.68	21.64
1	0.58	11.20	14.74	21.73
0continuous	0.56	11.59	15.22	21.97

3.5 Concluding Remarks

The presented finite element modelling techniques are an accurate means of calculating static deformation, modal characteristics, and time-history responses of beam-and-deck floors constructed from lightweight materials such as glulam. This supports application of contemporary vibration serviceability performance-related engineering design criteria that typically require estimation of small-amplitude static deflection caused by a concentrated vertical force and low-order modal frequencies of systems. A recurring theme throughout the present discussion is the need to calculate system-level stiffness coefficients associated with effects of a concentrated vertical force, or modal characteristics that define low-order natural frequencies by using models that realistically represent features of a particular system. Using simple closed-form formulas to predict static displacement and modal frequencies can result in unacceptable design inaccuracies. Consequently, the authors recommend use of accurate numerical structural analysis tools, rather than such formulas.

Currently there are no vibration serviceability guidelines established specifically with beam-and-deck systems in mind. The discussed finding that for such systems several lowest-order-modal frequencies depend closely on beam flexural rigidities indicates that the effects modal frequencies have on amplitudes and intensities of dynamic motions should not be ignored. This militates against applicability of simplified design practices that consider only static stiffness of a system or that in combination with the fundamental natural frequency.

Much current attention is being directed by researchers and structural code development committees toward creating generalized guidelines that will lead to greater surety that floors constructed from any materials by using any structural arrangement will function as intended in terms of vibration serviceability. The work reported in this paper is intended to help provide engineers with knowledge necessary for reliable application of existing or future guidelines having that purpose.

3.6 Acknowledgement

Financial support for the work reported was provided by the Canadian Natural Sciences and Engineering Research Council and the University of Ottawa.

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4 Chapter 4

VIBRATION CHARACTERISTICS OF A LARGE GLULAM BEAM-AND-DECK ELEMENT FLOOR SYSTEM

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(Submitted to ASCE, Journal of performance of constructed facilities, sent in revised format
28/6/2017)

4.1 Abstract

Discussion addresses experimental and analytical investigation of vibration characteristics of a large glued-laminated-timber (glulam) beam-and-deck system with office floor loadings. The system layout is complex with multiple bays supported by glulam columns. Ambient Vibration Test (AVT) was carried out before floor topping layers were added, and after occupation of the building. Addition of topping layers, and presence of office workers and furniture strongly influenced the fundamental frequency and damping characteristics of the system. Detailed Finite Element (FE) models were created to investigate how modelling assumptions affect accuracy of predicted modal characteristics. It was found to be accurate; FE analyses must be realistic to a degree that exceeds ‘typical’ design practice. Controlled walking tests indicative of motion levels associated with normal office activities were conducted. Those indicated the particular floor, and possibly other large glulam beam-and-deck floors, is not prone to vibration serviceability problems. Discussion addresses significance of test observations and avenues for improving vibration serviceability design practices.

Keywords: Beam, deck element, design, finite element, floors, glulam, simple formulas, vibration serviceability

4.2 Introduction

Contemporary engineering design practices aimed at avoiding construction of unserviceable lightweight timber floors address out-of-plane oscillatory motions caused by human footfall and other impacts occurring under normal building occupancy conditions (Weckendorf et al. 2015). Some timber design codes direct engineers to apply performance criteria specified in other documents. Others specify engineering formulas for calculating motion responses created by defined excitations and recommend associated limits on those responses. ‘Design of timber structures. Part 1-1: General - Common rules and rules for buildings’ is widely regarded as the primary example of the latter (CEN 2004). That document, commonly referred to as Eurocode 5, adopts a combination of criteria that address minimum acceptable fundamental natural frequency (8 Hz), maximum acceptable static displacement, and maximum velocity of a floor surface resulting from a unit impulsive force (1 Ns). The limitation on scope of application of those criteria is they only apply to rectangular plan floors in residential building having closely spaced joists overlaid by mechanically attached subflooring. An additional scope limitation is that all floor edges should be simply supported. Eurocode 5 scope limitation reflects those of the supporting research study (Ohlsson1988). Unfortunately such scope of application limitations are common for vibration serviceability design practices for lightweight timber floors across construction methods and building occupancy situations (CEN 2004; CSA 2014; CWC 1997; NRC 2015; ATC 1999; Allen and Murray 1993; Chui 1988; Dolan et al. 1999; FPL 2010; Hu and Chui 2004; Hu and Gagnon 2011; Onysko 1985; Smith and Chui1988; Toratti and Talja 2006).

Although scopes and technical underpinnings of design code and other design practice recommendations is variable (ranging from empirically calibrated methods, to ones with some level of dynamic analysis basis) there is a recurring commonality. The recurring commonality is basing design performance decisions, in part or in whole, on prediction of the fundamental natural frequency of a floor system (f_1), plus in some cases also the maximum displacement caused by concentrated gravity force of 1 kN (d_1) (e.g. CWC 1997; Onysko 1985; Hu 2000, 2013; Hu and Gagnon 2011; Weckendorf et al 2015).

Past investigators have carried out field assessments of vibration serviceability of timber floors. Weckendorf et al. (2014) assessed dynamic responses of joisted floors in four-storey timber light-framed apartment buildings. They found nominally identical elevated floors at different levels in a superstructure responded to dynamic force disturbances in a remarkably similar way. They also found flanking motion transmissions people can sense, occur vertically and horizontally within building substructures, and nonstructural walls had small effect on modal characteristics of joisted floors that support them. Jernero et al. (2015) compared dynamic response of unidirectional rib-stiffened cross-laminated-timber (CLT) floor slab-modules that were either structurally isolated (uncoupled) or were installed within a superstructure (coupled). Only weak differences were found in effective modal stiffnesses and mass characteristics of uncoupled and coupled modules. By contrast, coupled modules had greatly increased modal damping ratios. Supported partition walls in an L-shaped on plan strongly affected modal characteristics of coupled modulus.

Most design guidelines (CEN 2004, NRC 2015, ATC 1999) are based on assuming fundamental frequency of timber floors should be above 8 Hz as this is the frequency threshold below which human preceptory organs resonate (Smith 2003). By implication glulam beam-and-

deck floors and possibly other light weight timber floors with fundamental frequencies below 8 Hz might be suspected to be inherently prone to lively responses to impact forces.

This paper addresses experimental and analytical investigations of serviceability level vibration characteristics of a large glued-laminated-timber (glulam) beam-and-deck system with office floor loadings. Such floors are relatively new and few and little is known about their performances. Glulam beam-and-deck floors have relatively widely spaced parallel glulam beams which support mechanically attached flat wise oriented glulam deck elements, as illustrated in Fig.4.1.



Fig. 4.1. Glulam beam-and-deck element floor system supported on glulam columns

This paper has the following objectives:

- (1) To investigate effects of nonstructural elements like concrete overlay and imposed objects like furniture have on vibration characteristics and motion levels of a large glulam beam-and-deck element floor system.

- (2) To develop a finite element (FE) modelling approach that can accurately predict mode shapes and modal frequencies of large glulam beam-and-deck element floor systems.
- (3) To assess ability of analytical formulas to predict the fundamental natural frequency of an open-plan glulam beam-and-deck element floor systems having long-spans and multiple bays.

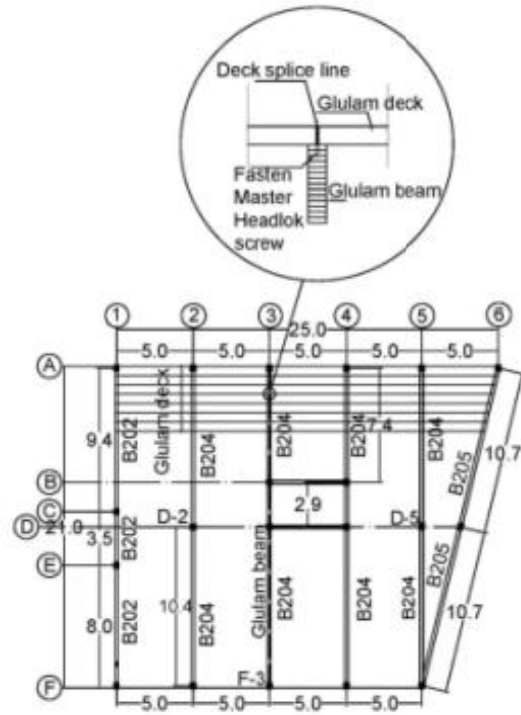
4.3 Study Building

The beam-and-deck element floor evaluated is located in the East Wing of the Mosaic Centre in Edmonton, Alberta, Fig. 4.2. The Mosaic Centre was opened in February 2015 as a showcase net-zero energy commercial building. It embodies advanced construction technologies suited to continental northern climates. The East Wing has three storeys (i.e. 2 two elevated floor levels) and is connected to a two-storey West Wing by a staircase well, in a manner that achieves structural and fire separation. Edmonton has an average annual ground snowfall exceeding 1.2 m, meaning roof and floor design live loads created dominant design forces determining sizes of superstructure elements. Effects of vertical loads on the study floor are resisted by a framework of glulam posts and beams. The superstructure has five parallel and one obliquely arranged primary beams at each elevated floor level, Fig. 4.3. Effects of wind and other lateral design forces are resisted by timber shear-walls. The dynamic response characteristics of the floor system of the second storey (i.e first elevated floor) of the East Wing were evaluated. When this paper was written that floor was one of the world's largest glulam beam-and-deck systems.



Fig. 4.2. Mosaic Centre, Edmonton, Alberta

As shown in Fig. 4.3(a), the floor system has trapezoidal overall plan shape and semi-irregular structural supports. Those and other features ensure the dynamic response characteristics are complicated. Maximum beam spans are 10.8 m (i.e. maximum column spacing) with parallel beams set 5.0 m apart. Consequently, the dominant nominal span for decking elements is 5.0 m. Beams have simple end supports achieved by face mounting them to columns, Fig. 4.4. Structural glulam element dimensions and grades are summarized in Table 4.1. Deck elements are 130 mm thick and laid in the pattern illustrated in Fig. 4.3a. They are mostly attached to glulam beams using two rows of 203 mm long by 6.5 mm diameter Fasten Master Headlock™ screws spaced at a minimum of 75 mm. Such screws anchor the deck to and prevent deck elements from bouncing relative to beams during oscillatory floor motions. Adjacent deck elements have a 6 mm expansion gap between them. There is also an expansion gap coincident with frame 3, as shown in Fig 4.3a. Typical bolted connections between beams and columns are shown in Fig 4.4.

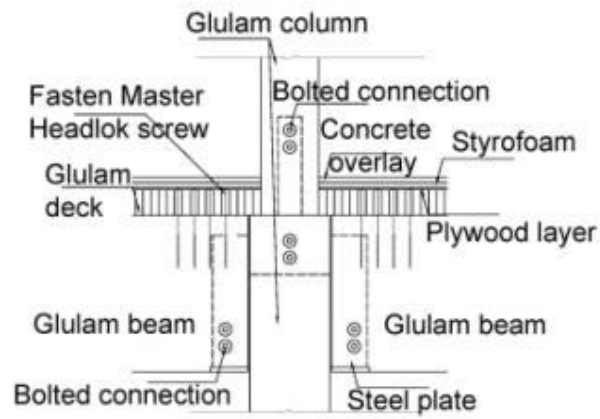


(a)

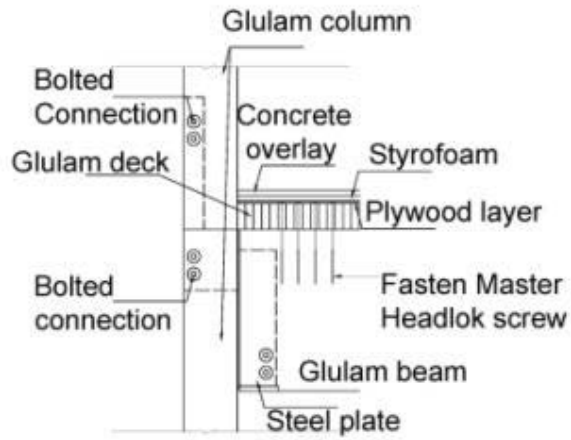


(b)

Fig. 4.3. Study floor (a) Structural layout (b) Underside view during construction



(a)



(b)

Fig. 4.4. Sample beam-to-column connections: floor of second storey (a) Interior framework connection: location D-2 (b) Exterior framework connection: location F-3

Table 4.1. Details of structural glulam elements

designation *	Width (mm)	Height (mm)	Grade**
Beams: <i>Douglas fir</i>			
B202	265	643	24f-E
B204	215	795	24f-E
B205	215	529	24f-E
Columns: <i>Douglas fir</i>			
1 st storey:			
D-3	215	377	16C-E
D-2	215	264	16C-E
2 nd storey:			
D-3	215	377	16C-E
D-2	215	264	16C-E
Deck elements: <i>Spruce-Pine-Fir</i>			
WD	130	603	No. 2

* Element designations match Fig. 4.3(a).

** Grades conform to CSA Standard O86 (CSA 2014).

Nonstructural elements are added to the floor consisting of nominally 13 mm thick plywood sheathing attached to the tops of deck elements, 25 mm Styrofoam rigid insulation, and 30 mm of poured concrete topping. The concrete topping is 32 MPa concrete with fiberglass micro-reinforcing strands and mean density 2350 kg/m³. Although all regarded as nonstructural layers of topping materials created continuity between bays of the floor, and add significant modal mass, stiffness and damping. Effects of those topping and occupancy loads (e.g. office furniture and people) on the dynamic response of the floor system are evaluated below.

4.4 Dynamic Tests

4.4.1 Modal parameter characterization

Ambient Vibration Tests (AVT) were made on the study floor in its uncompleted and occupied forms, Table 4.2. AVT is a powerful and accurate approach for *in-situ* description of modal characteristics of large structural systems (e.g. Brincker and Ventura 2015, Rainieri and Fabbrocino 2014). Dynamic response was measured using grids of measuring points, as illustrated in Fig. 4.5(a). Response measurement points were selected based on preliminary FE analysis, to ensure reliable detection of low-order modes and accurate determination of modal response characteristics. In each series of tests a Tromino™ data acquisition system was used to collect floor motion data. One Tromino sensor captured motions at a constant position as the master location marked by a large dot in Fig. 4.5(a). Roving sensors, shown with small dots in Fig. 4.5(a), measured responses at other grid points, with many sets of measurements taken for each roving sensor locations. The duration for each set of measurements was 120 seconds.

Table 4.2. Tested floor conditions

Uncompleted floor	Without plywood, Styrofoam and poured concrete overlays
Occupied Floor	With nonstructural overlays (plywood, styrofoam and poured concrete overlays), installed office furniture and equipment as well as seven people presented on the floor

During uncompleted floor stage tests only the two people making AVT measurement were presented on the floor. The floor was set into motion by the forcing effect of footfalls produced by one test crew member marched on its surface. Occupied floor stage tests were carried out during a normal working day with office furniture and equipment installed, and seven office workers and the

two members of the test crew on its surface. In that case the excitation source was a 5 kg medicine ball dropped onto the floor surface to create impacts. A test crew member was walking around the floor and dropping the ball from about 1 m at irregular intervals. Such impact loads is applied to ensure a low noise-to-signal ratio as recommended by various references. (Cantieni and Biro 2005; Ramos et al. 2010; Masciotta et al. 2017).

Data was preprocessed before conducting modal analysis to using high pass filtering to remove the static component at 0.0 Hz. Modal analysis was performed using the Enhanced Frequency Domain Decomposition (EFDD) technique, as implemented by the Artemis Extractor 5.3 software tool (SVS 2011). In EFDD the autocorrelation function is used to extract modal frequencies and damping ratios. First the single degree of freedom (SDOF) spectral density functions (SDOF bell) are calculated around a resonance peak using Fast Fourier Transform (FFT) of the measured signals. The corresponding SDOF autocorrelation function was then obtained using an Inverse FFT (IFFT) of identified SDOF bell function. The associated natural/modal frequency and equivalent viscous damping are obtained from the estimated zero crossing time and logarithmic decrementation of the corresponding autocorrelation function (Brincker and Ventura 2015). EFDD was developed to overcome limitations of normal Frequency Domain Decomposition (Jacobsen et al. 2006).

Fig. 4.5 (b) shows the spectral density function for study floor. The boxes indicate modal frequencies extracted using single degree of freedom bell shape functions by the EFDD technique. Extracted mode shapes, modal frequencies and effective modal damping ratios for the first three modes of the uncoupled (incomplete) floor are shown in Fig. 4.6.

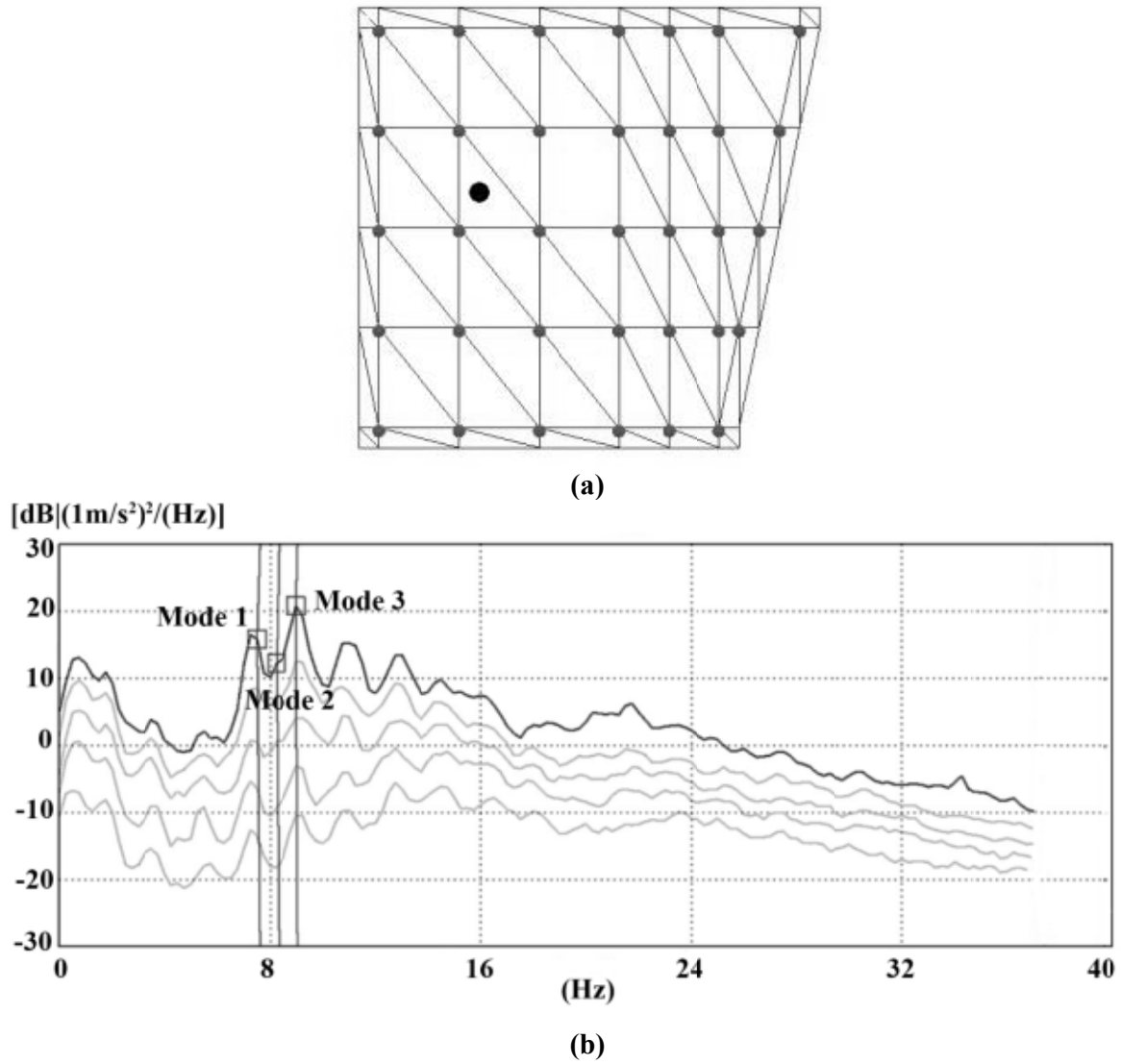


Fig. 4.5. AVT measuring grid and spectral density function (a) Grid of measuring points during AVT (large dot = master location) (b) Spectral density function

Vibration tests after completion and occupation of the building resulted in an extracted fundamental natural frequency (f_1) of 5.0 Hz and a corresponding modal damping ration (ζ_1) of 10.0 %. This compares with $f_1 = 7.6$ Hz and $\zeta_1 = 2.7\%$ for the uncompleted building. Characteristics of

other modes were not as well defined, but the data indicated second and third modal frequencies were reduced and associated modal damping ratios much increased relative to values for the uncompleted building. This clearly demonstrates installation of nonstructural building fabric and occupation of the building substantially increased modal masses and modal damping, but did not increase modal stiffnesses significantly.

As already noted in the Introduction, design practices for lightweight timber floors are often predicted on the notion that floor with f_1 values below 8 Hz are prone to unacceptable vibration serviceability performance (Weckendorf et al 2015). The marked increases in damping after building completion and occupation is attributable to increased frictional damping in the floor itself and damping added by the presence of human bodies and objects like furniture the floor supports. Similar large increases in damping are commonly reported in the literature (Weckendorf et al. 2015).

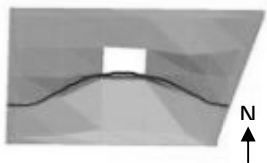
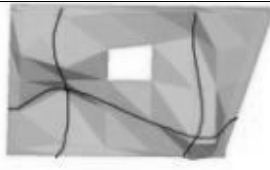
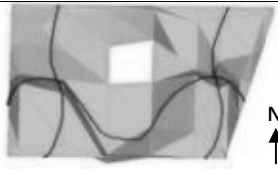
Mode	1	2	3
Natural frequency (Hz)	7.6	8.3	9.1
Modal damping ratio (%)	2.7	2.1	2.5
Mode shape			

Fig. 4.6. Measured modal characteristics and mode shapes of the incomplete floor(i.e. without plywood, Styrofoam and poured concrete overlays).

4.4.2 Walking excitation

Replicated walking tests were performed to measure the acceleration levels while a person walked on the study floor at normal walking speed (i.e. stepping frequency of about 2 Hz). During a test seven people other than a test crew were present but did not walk on the floor. The test was conducted three times for each of three routes of between 8 m and 11.5 m. Six equally spaced Tromino sensors recording acceleration levels along a route as shown in Fig. 4.7. Chosen routes excited the floor system primarily in one of the open portions of the primary office space, in a manner representative of how workers transit them. Given the measured fundamental frequency of the floor was 5.0 Hz, it was suspected in advance that at least the third harmonic of the forcing frequency would resonate the floor. Measured signals were preprocessed using Matlab software (TMWI 2013) to subtract the average of the signal (remove offset), also a low pass filter was applied to remove the frequency content above 40 Hz. The applied signal filtering matched the approach of other researchers who made similar measurements (AISC 1997; Ohlsson 1988). Table 4.3 summarizes averaged peak acceleration levels for each route. Also shown in Table 4.3 are tolerable Equivalent Sinusoidal Peak Acceleration (ESPA) limits suggested by the American Institute of Steel Construction (AISC 1997; Davis et al. 2014; Zhou et al, 2016):

$$a_{rms}(t) = \sqrt{\frac{1}{N} \sum_{i=1}^N a_i^2} \quad (4.1)$$

where $a_{rms}(t)$ is rolling rms acceleration at time (t), a_i is acceleration response vector array, and N is the number of arrays in the interval from $t-1$ to $t+1$. Calculated ESPA values are obtained by

multiplying the maximum rolling rms value (from Eq. 4.1) by $\sqrt{2}$ and expressed as a percentage of acceleration due to gravity (%g).

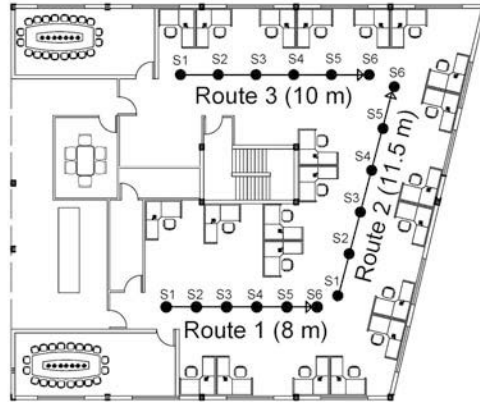


Fig. 4.7. Walking routes for assessment of acceleration response

As Table 4.3 shows, the maximum estimated ESPA value was 0.044 % g, which is much less than the limit 0.5% g for office floors according to AISC (1997). Typical time history response and rolling RMS obtained from selected sensors in various routes is shown in Fig. 4.8. A survey conducted by the test crew revealed no adverse comments from office workers about floor performance. For the particular combination of floor construction method, floor pan and building occupancy situation the walking test results and lack of adverse building occupant comments clearly indicate there would be no validity to simple prescriptive design requirements like ensuring f_1 is > 8 Hz, i.e a so-called high frequency tuning approach would be invalid. The same conclusion was reported in the assessment of vibration performance of foot bridges under walking load (Pimentel et. al 2001).

High frequency tuning has been proposed by US researchers (Dolan et al. 1999) who suggested the minimum f_1 of 15 Hz for unoccupied and 14 Hz for occupied light weight joisted timber floors. Criteria suggested by other researchers (e.g Hu 2007, Hamm et al. 2010) include combined tuning of f_1 and d_1 . Newly adopted criteria based on Vibration Dose Value (VDV) in British Standard Institute (BS 6472 (BSI 2008)) and International Organization for Standardization (ISO 10137 (ISO 2007)) as well as ESPA (AISC 1997) have been suggested as better alternatives. Various references support the idea that FE analysis tools are reliable tools for predicting actual dynamic response of floors (Pavic and Raynolds 2002, Bachmann and Ammann 1987, Setareh 2010). It is arguably fair to say that modern scientific opinion is converging towards rejection of notion that simplified design practices like control of f_1 are reliable ways on ensuring acceptable vibration serviceability performance. Results presented here may not mean that under other building use scenarios, or if the construction feature (Like non-structural topping layers) differed, the same would necessarily be true. The matter demand further investigation.

Table 4.3. Average peak acceleration, and ESPA values (%g)

Sensor	1	2	3*	4	5	6
Route 1						
Peak acc (av.)	0.072	0.072	-	0.065	0.063	0.065
ESPA (av.)	0.032	0.032	-	0.031	0.030	0.031
Route 2						
Peak acc(av.)	0.073	0.119	-	0.134	0.072	0.073
ESPA (av.)	0.035	0.044	-	0.037	0.031	0.032
Route 3						
Peak acc (av.)	0.069	0.072	-	0.060	0.072	0.061
ESPA (av.)	0.030	0.031	-	0.031	0.030	0.028

*Data for sensor 3 is no reported because the signal was judged to be too noisy.

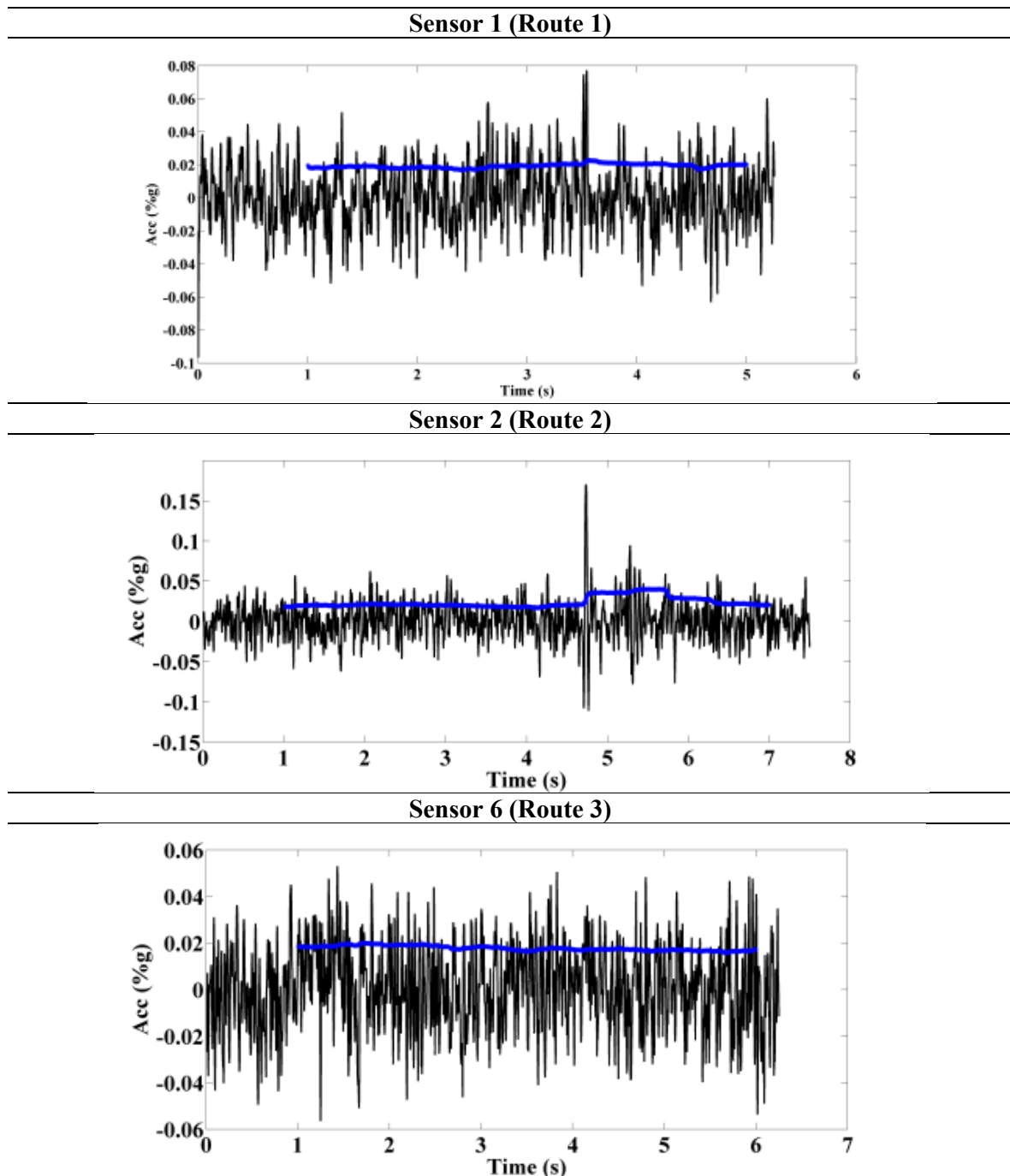


Fig. 4.8. Typical time history response and rolling RMS for various sensors obtained from walking test

4.5 Finite Element Analyses

4.5.1 Modeling approach

FE models were created using the Abaqus software package (SIMULIA 2012). Glulam beams and deck elements were modeled as linear-elastic orthotropic materials with elastic constants defined by parallel to length, width and depth axes of material symmetry (denoted axes 1, 2, 3). Assumed elastic constants are listed in Table 4.4. That tabulation also gives glulam densities, and the stiffness coefficient characterizing the slip responses of screw fasteners attaching deck elements to beams ($k_{\text{deck-to-beam}}$).

Table 4.4. Elastic constants and density values used in FE models

Constant*	Value (units)	Source
Glulam beams:		
E_1	12.8 GPa	CSA O86 (2014)
G_{13}	0.8 GPa	FPL(2010)
Density	530 kg/m ³	Manufacturer's literature
Glulam deck elements:		
E_1	10.0 GPa	Ebadi et al (2016)
G_{12}	0.7 GPa	Ebadi et al (2016)
Density	490 kg/m ³	Ebadi et al (2016)
Glulam Columns:		
E_1	12.4 GPa	CSA O86 (2014)
G_{13}	0.8 GPa	FPL(2010)
Density	530 kg/m ³	Manufacturer's literature
Mechanical fasteners:		
$k_{\text{deck-to-beam}}$	540 kN/m	Ebadi et al (2016)

* E_1 = elastic moduli, G_{12} , G_{13} = shear moduli,

Beams were modeled using a three-node quadratic type B32 Timoshenko beam elements which include shear deformation and rotary inertia effects. This choice is because the beams are relatively deep. Deck elements were modeled using orthotropic reduced integration type S4R shell elements with hourglass control. These elements were chosen because the deck is not very thick. Selected elements have three translational and three rotational degrees of freedom per node. Screw connections between deck and beam elements were modeled using three orthogonally arranged linear spring elements located at each screw position (i.e. two horizontal and one vertical). Deformation of each spring was restricted by two rigid link elements, with the length of the upper link being half the deck-element thickness and length of the lower link elements half the beam depth. This prevented vertical separations between deck elements and beams. Link elements were assigned zero mass.

As already discussed, contemporary design codes and design practices for timber floor systems are based on assuming motions are dominated by flexural modes. It is also often assumed supports are simple (Weckendorf et al. 2015). Beams of the study floor were designed to each behave as pinned-ended members attached to faces of columns, Fig. 4.4. Consistent with that, the reference condition for analyzing the system was to assume beam ends are swivel connected to columns, i.e. no translation but fully free to rotate relative to support surfaces. Columns were modeled using three-node quadratic Timoshenko beam elements. Those were incorporated both below and above the floor level in a manner that permits only axial deformation. Assigned upper and lower column lengths were 3.75 m and 3.9 m respectively for first and second storeys, corresponding to physical dimensions of the building. Only the masses of columns were considered in frequency analysis, with overburdening mass of the roof neglected. This representation may not be truly realistic but create a reference point for investigation of effects various departures from

those conditions have on FE predictions of modal characteristics of the system. Departures investigated altered the axial stiffnesses of columns, and encastered beam ends (referred to below as fixed ends). In FE models, splice lines and gaps between adjacent deck elements were replicated by not interconnecting adjacent shell elements of the deck at those locations. The floor edges were modelled as having rigid pin supports (i.e. no horizontal or vertical translation) at locations of supporting shear-walls (i.e. walls on the plan perimeter or encasing the building core). A 1.0 kN/m force was applied at free perimeter wall locations to simulate influences supported gravitational mass. In practical terms the model representations permitted investigation of sensitivity of mode shapes and frequencies to column motions and effects of assuming continuity between bays of the floor system.

The Lanczos eigenvalue solution method was used to estimate modal frequencies. That method efficiently extracts modal frequencies up to about 50 Hz, which was the range chosen for analyses, based on the 40 Hz analysis filtering frequency recommended by Eurocode 5 for lightweight joisted floors (CEN, 2004). Automatic meshing was applied using the Abaqus preprocessor. A global seed mesh size of 0.25 m resulting in convergent solutions.

4.5.2 Uncoupled response of bare system

Figs. 4.9 to 4.12 show FE model predictions of modal frequencies and mode shapes based on various beam end condition and column stiffness presumptions. Case 1, corresponds to beam ends allowed to swivel on top of rigid columns, which corresponds to a representation likely to be adopted by design engineers. Comparison of FE predicted with test derived results shows good agreement between low order modal frequencies (f_1 : 7.6 vs. 7.6 Hz, f_2 : 8.2 vs. 8.3 Hz, f_3 : 9.0 vs. 9.1

Hz). Correspondingly, predicted low order mode shapes replicate test derived one quite well (Fig. 4.9).

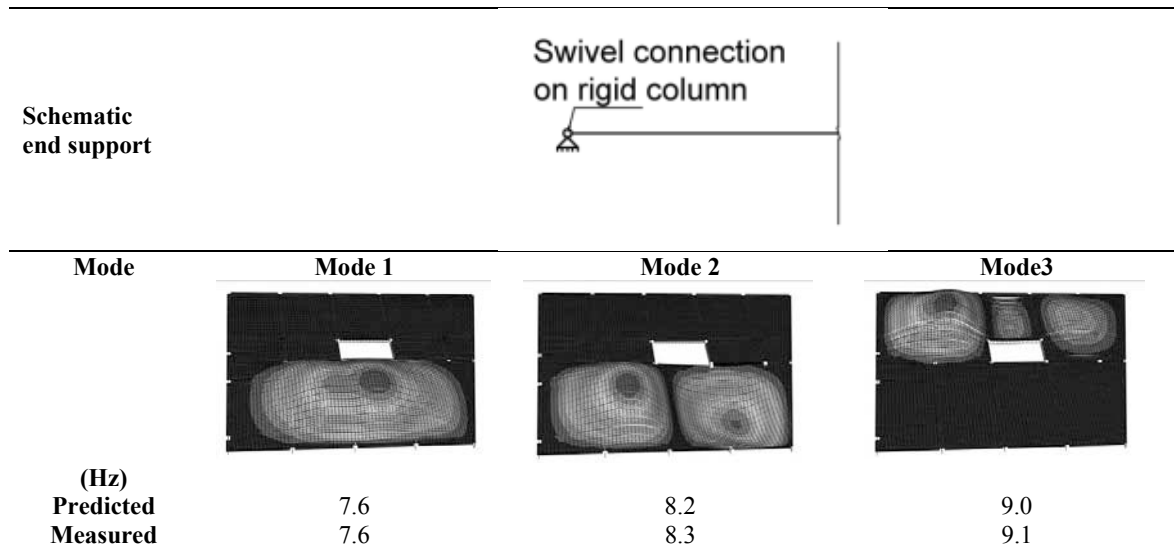
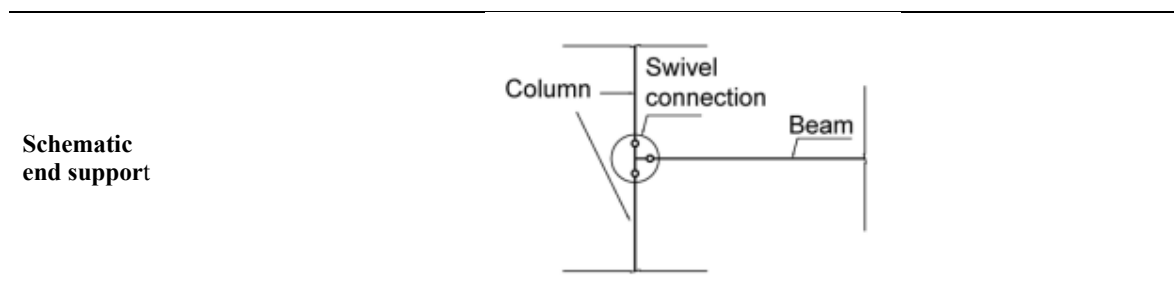


Fig. 4.9. Predicted modal frequencies and mode shapes: case 1

Case 2, where beam ends are assumed to be swivel connected to flexible columns improved the agreement between model and test results for f_1 (7.3 vs. 7.6 Hz). However, it produced poorer predictions of f_2 and f_3 (mode 2: 7.9 vs. 8.3 Hz, mode 3: 8.6 vs. 9.1 Hz). This is attributable to discrepancies in both modal stiffnesses and masses (Fig. 4.10). Case 1 and Case 2 models predicted almost identical mode shapes.



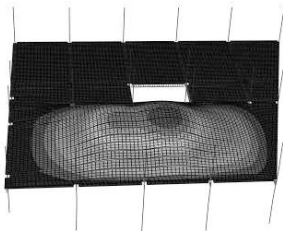
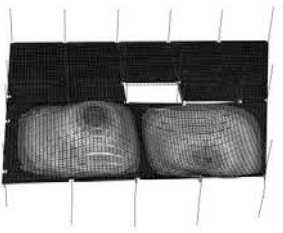
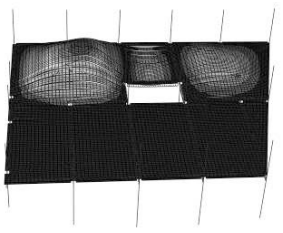
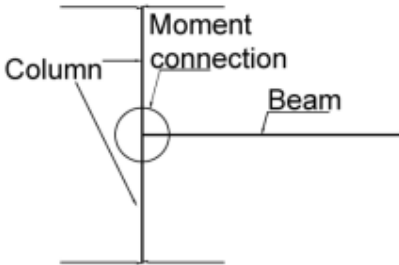
Mode	Mode 1	Mode 2	Mode3
			
(Hz)			
Predicted	7.3	7.9	8.6
Measured	7.6	8.3	9.1

Fig. 4.10. Predicted modal frequencies and mode shapes: case 2

Case 3 illustrates the maximum possible influence of altering the superstructure framing system by making it act as a moment frame work with columns sway prevented. This altered mode shapes and caused the three lowest modal frequencies to tightly cluster (9.0, 9.6 and 9.9 Hz). From a vibration serviceability performance perspective this suggests a tendency to amplification of motions sensed by building occupants. If the modelling assumptions were valid it would mean application of design criteria that focus solely on the fundamental modal response could not reliably predict dynamic responses of such large beam-and-deck element floor systems (Ussher et al. 2016). Assumptions underpinning Case 3 did not match test results well (Fig. 4.11).

Schematic end support				
	Mode	Mode 1	Mode 2	Mode3

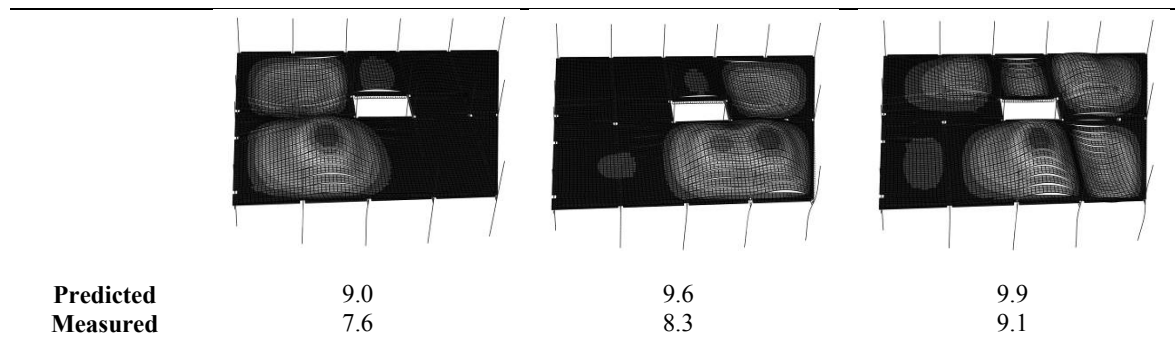
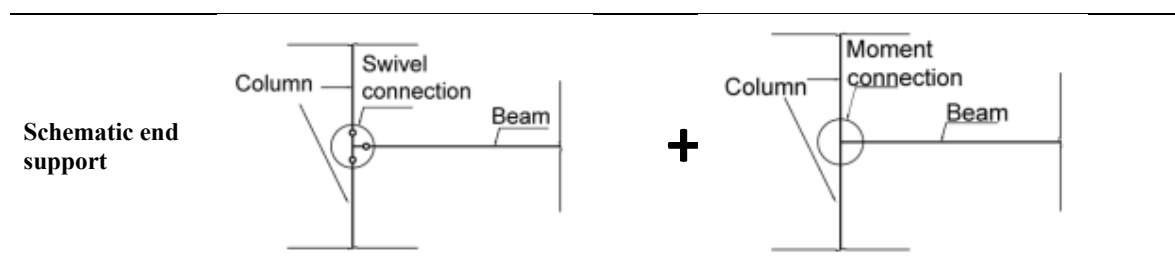


Fig. 4.11. Predicted modal frequencies and mode shapes: case 3

Case 4 illustrates selective rather than blanket incorporation of beam end-fixity into the system. The chosen beam fixity locations enabling moment transfers at only column locations D-2 and D-5 (Fig. 4.3). Comparing test results reveals good correspondence between mode shapes and almost exact correspondence of the three modal frequencies (i.e. 7.4 vs. 7.6 Hz, 8.2 vs. 8.3 Hz, and 9.2 vs. 9.1 Hz) (Fig. 4.12). This suggests interior beam end connections did transfer moments between beams meeting at interior columns. In other situation the truest representation of interior end fixities of beams would vary depending on design decisions and how designs are implemented on construction sites. The four considered cases assist cognizance that predictions of modal frequencies and mode shapes are sensitive to choices engineers make undertaking analyses. It can alternatively be interpreted as showing engineers have great ability alter how particular systems vibrate through their design choices of structural/construction details.



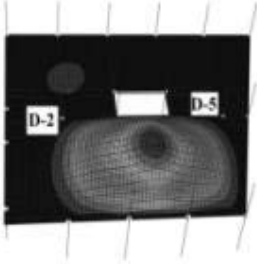
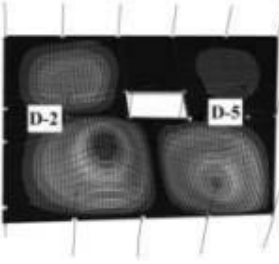
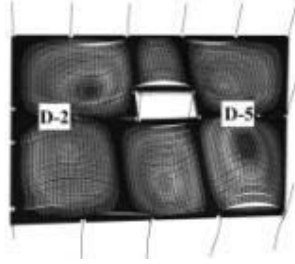
Mode	Mode 1	Mode 2	Mode3
			
(Hz)			
Predicted	7.4	8.2	9.2
Measured	7.6	8.3	9.1

Fig. 4.12. Predicted modal frequencies and mode shapes: case 4

4.6 Discussion

In design practice engineers often use analytical formulas to estimate behaviours of large substructures like the study floor. They could, for example, design floors as though they behave as isolated bays defined by column spacing in parallel and normal to beam span directions. Simplified analytical formulas exist for predicting fundamental natural frequencies of simply-supported one-way or two-way spanning lightweight rectangular joisted floors (e.g. Hu 2000; Hu and Chui 2004; Toratti and Talja 2006). Those formulas are based on how isolated simply-supported beam vibrate; assume a half-sine wave mode shape; and substituting floor mass per unit area and flexural rigidity per unit floor width for beam mass per unit length and beam flexural rigidity. The following equation from Eurocode5 (CEN 2004) is such an equation:

$$f_1 = [\pi / 2l^2] \bullet [EI / m]^{1/2} \quad (4.2)$$

where EI is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction, m is the mass per unit floor area, and l is the floor span. In the present study

building equation (4.2) may be applied assuming span (L) equals the lengths of a rectangular bay defined by the maximum column spacing (10.8 m), and EI is the principal flexural rigidity of the floor. That results in an estimated value of f_1 of 7.8 Hz. This compares with test and FE model with matched assumptions values of 7.6 and 7.6 Hz. Whether the simple formula value is sufficiently accurate will depend on how it will be used subsequently. In other words, there is uncertainty regarding the assumption of the beam length (e.x clear span, full span or design span), assumed material properties including density, modulus of elasticity and boundary condition which might cause different answer in comparison with measurements.

As FE analyses show the type of connections a glulam beam-and-deck floor system has significant effect on mode shapes and corresponding modal frequencies. It is therefore important to properly select modelling methods, with emphasis on one verified against laboratory or field test results. This same observation has been made for other types of large lightweight timber floor systems (e.g. Ussher et al. 2016; Weckendorf et al. 2014; Weckendorf and Smith 2012).

As already mentioned but not fully discussed, the maximum average ESPA value resulting from walking tests was much less than the limit proposed by AISC Design Guide (AISC 1997). This is attributable to the high level of damping associated with presence of nonstructural elements (i.e. plywood, rigid insulation and poured concrete topping), and presence of furniture and office workers (AISC 1997; Weckendorf et al 2015).

Various proponents have recommended necessity of creating floors having fundamental natural frequencies not less than a prescriptively defined value, with > 8 Hz being a popular choice (e.g. CEN 2004; Chui and Smith 1990; Dolan et al 1999; Hu and Chui 2004; Ohlsson 1998; Smith and Chui 1988; Weckendorf et al. 2015). Such recommendations stand alone or go hand in hand

with vibration serviceability assessment criteria are abstractions of how humans sense and tolerate oscillatory motions. Embedded in an 8 Hz or similar minimum f_1 requirement is an assumption that human sensitivity correlates with the likelihood frequency contents of motions will cause internal human body sensory organs to resonate (Ohlsson 1982; Weckendorf et al. 2015). Whether that concept is sound is doubtful however. The present walking test results support the notion that at least some proposed design criteria and methods are founded on wrong concepts.

The authors believe only design criteria and methods based on prediction of floor surface motions created by impacts typical of particular building occupancy situations can yield consistently satisfactory design solutions. The Eurocode 5 (CEN 2014) method for joisted floors is predicated on that philosophy and can be the foundation for creating something similar that applies to all types of lightweight floors, rather than only certain types of joisted floors. Ongoing activities of Canadian, Eurocode 5 and other timber design code committees are actively addressing the above, with support from groups like the International Network on Timber Engineering Research.

4.7 Conclusions

Field investigation and numerical analysis studies demonstrate relatively large glulam beam-and-deck element floor systems have complex dynamic responses. Finite element analysis show low order mode shapes and modal frequencies are strongly dependent on assumptions about how beams ends are coupled to supporting columns. Tests and model investigations demonstrate such floors can exhibit clustered low order modal frequencies. Traditional wisdom would suggest such floors might be prone to creation floor motions unacceptable to building occupants. However, that is not the case for the study floor, as illustrated by low levels of motion measured during controlled walking tests and lack of building occupant complaints. This and other evidence indicated

simplified design analysis methods and vibration serviceability design methods proposed for other types of lightweight floor systems can be invalid for glulam beam-and-deck floors. Presented results and discussion can support ongoing international efforts to define robustly reliable dynamic serviceability design concepts and methods.

4.8 Acknowledgements

Financial support was provided by the Canadian Natural Science and Engineering Research Council (Grant T2-7-C3/NEWBuilds). We also acknowledge Mr. Andre Lema from the Western Archrib company, and the building owners (Cuku's Nest Enterprises Ltd) who provided technical data and access to study building. Ms. Maryam Ebadi is thanked for assisting with tests.

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5 CHAPTER 5

SUPPLEMENTARY INVESTIGATIONS AND DISCUSSION

This chapter elaborates supplementary investigations and discussion that builds on Chapters 2 to 4.

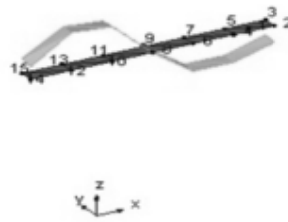
5.1 Chapter 2 (Paper 1): Vibration serviceability of glulam beam-and-deck floors

5.1.1 Dynamic response of deck elements

An isolated single glulam deck element with a span of approximately 5 m attached to a wooden frame using 4 HecoTopix screws was tested with the measured fundamental natural frequency (f_1) being 6.4 Hz. Adding a middle support, Fig. 5.1 (a), increased f_1 to 26.6 Hz, with the matched FE model analysis predicting 26.4 Hz. The approximate fourfold change in f_1 and the mode shape changing from a half sinewave to a full sine wave parallel to span, Figs. 5.1 (b) and 5.1 (c), matches normal expectations for simply supported beams. Nominally the mid-span supported case corresponds to support conditions for a deck element in Floor 0, which had an f_1 of 11.1 Hz (Table 2.3, Chapter 2). This is because the fundamental mode response of the floor is dominated by the flexural behaviour of beam elements rather than deck elements.



(a) Glulam deck element with middle support



(b) Mode shape obtained from AVT



(c) Mode shape obtained from FEM

Fig. 5.1. Isolated deck element test and analysis results

The effect of overburden force simulating the mass of an upper storey(s) on floors was investigated (Fig. 5.2). The test simulated a 5 m glulam deck element sandwiched between walls of adjacent storey(s) in a building superstructure was laid over two wood frame supports having a load cell and a hollow steel section. A “Z” shape steel section was placed over the glulam deck with force applied to it by two threaded rods using torqued nuts on the rods and measured by a load cell. Table 5.1 shows the relationship between the applied overburdening force and the f_1 of the glulam deck element. As can be seen applying overburdening force tends to encaster deck elements at supports. This implies support conditions will influence modal frequencies of floors in completed buildings.



Fig. 5.2 Effect of overburden force

Table 5.1 Effect of overburdening force on f_1

Applied overburden force (kN)	f_1 (Hz)
0	6.1
0.65	6.6
1.50	6.8
3.00	7.0

5.1.2 Comparison of experimental results with suggested design criteria

Table 5.2 compares outcomes of various vibration serviceability criteria suggested for design of lightweight timber floors with results of focus group ratings of laboratory built glulam beam-and-deck floors.

Table 5.2. Comparison between outcomes of suggested design criteria and focus group ratings

Criterion			Floor*			
Requirement	Scope	Author	0	1	2	3
$d_1 < \frac{8}{span^{1.3}}$	Joisted floors in small buildings	Onysko 1985	P	P	P	P
$\frac{f_1}{d^{0.44}} > 18.7$	Joisted floors in small buildings	Hu and Chui 2004	F	P	P	P
$\frac{f_1}{d^{0.7}} > 13$	CLT floor in residential and office buildings	Hu and Gagnon 2011	P	P	P	P
$f_1 > 14Hz$	Joisted floors in occupied buildings	Dolan et al. 1999	F	F	P	P
Subjective assessments by focus groups			F	F	F	M

* F = unacceptable, P = acceptable, M = marginal

Results in Table 5.2 indicate application of criteria suggested by Onysko (1985) and Hu and Chui (2004) for joisted floors, or by Hu and Gagnon (2011) for CLT floors would lead to consistent overestimation of acceptability of glulam beam-and-deck floors. The very simple pass or fail criterion based on f_1 suggested by Dolan et al. (1999) appears to work better in general, but still tends to overrate floor performances. The broad conclusion is however that attempting to apply subjective opinion based design criteria developed for other types of floor systems is a totally unreliable practice for beam-and-deck floors.

5.2 Chapter 3 (Paper 2): Finite element modeling and parametric study of glulam beam-and-deck floors

5.2.1 Comparison of FE model and explicit formula predictions

Fig 5.3 compares f_1 and d_1 values derived from test data, explicit formula of Chapter 2, and FE models of Chapter 3. The relevant explicit formulas are equations (2.2) and (2.3) of Chapter 2. The broad conclusion is that the test values of f_1 and d_1 predicted by either explicit formulas or FE models have accuracy engineers might well consider sufficient for design.

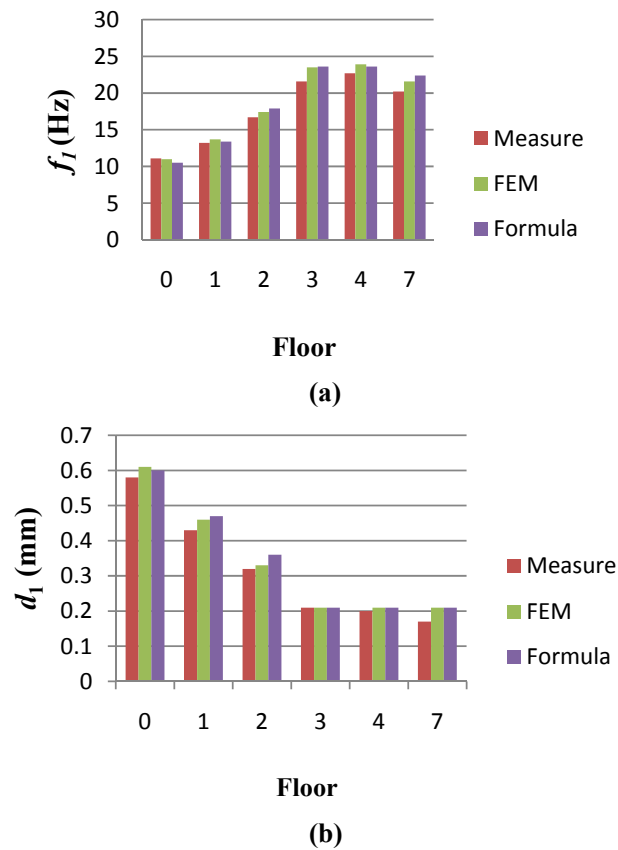


Fig. 5.3. Comparison of f_1 and d_1 value derived from test data, explicit formula and FE models (a) f_1 (Hz) (b) d_1 (mm)

5.2.2 Effect of floor boundary conditions

Verified FE model is used to predict the vibration performance of the floors with different boundary conditions. To define different boundary conditions, longitudinal and transversal movements of edge beams and deck elements were selectively restrained. Fig. 5.4 shows predicted mode shapes, and modal frequencies and masses for 5m by 5m floors with three beams for boundary conditions of 2, 3 and 4 edges simply supported. The 2 edges simply supported condition corresponds to Floor 0 in the test program. Fig. 5.5 shows modal information for the 5 m by 5 m floor 3 edges simply supported. Three edges simply supported means beam ends and one edge beam are simply supported along their lengths. Cases where the modal mass is denoted 0.0 kg correspond to situations where FE models predictions insignificant values. Those cases correspond to ones where anti-symmetry results in cancelling sub-mass motions. Modal frequencies are rounded to three significant figures as a recognition of realistic prediction accuracy.

Important points to note from Figs. 5.4 and 5.5 are:

- All mode shapes exhibit two way curvature (i.e. parallel and perpendicular to the beam span direction).
- All global mode shape cross-sections are symmetric about the across beams mid-span axis of the floor (i.e. axis coincident with mid-spans of beams).
- When only beam end edges or all four edges are simply supported all global and local mode shape cross-sections are symmetric or anti symmetric about the mid-width axis of the floor.
- When beam ends and one edge beam are simply supported global mode shape cross-sections are not symmetric about the mid-width axis of the floor.
- Local mode shapes are dominated by flexural deformations of deck elements, and have clustered modal frequencies (leading to what are sometimes referred to as shadow modes).

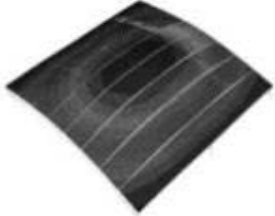
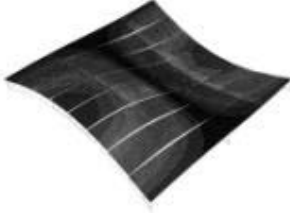
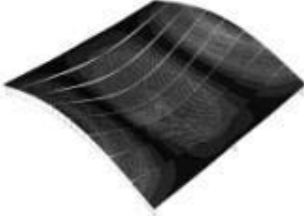
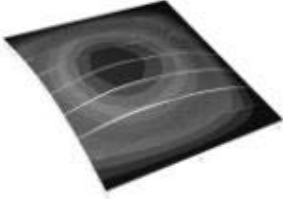
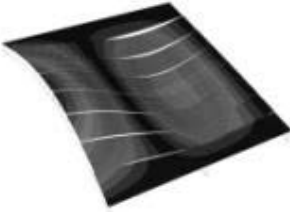
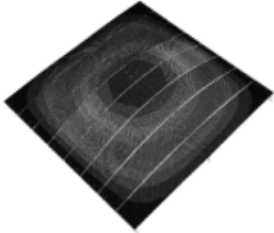
	Mode 1	Mode 2	Mode 3
	$f_1 = 11.0 \text{ Hz}$ (1 st first order mode)	$f_2 = 14.6 \text{ Hz}$ (2 nd first order mode)	$f_3 = 21.5 \text{ Hz}$ (3 rd first order mode)
2 ends supported			
Modal mass (kg)	929	0.0	55.4
	$f_1 = 12.3$ (1 st first order mode)	$f_2 = 17.7$ (2 nd first order mode)	$f_3 = 27.2$ Local mode
3 sided support			
Modal mass (kg)	793	0.61	0.0
	$f_1 = 12.6 \text{ Hz}$ (1 st first order mode)	$f_2 = 27.2 \text{ Hz}$ Local mode	$f_3 = 27.4 \text{ Hz}$ Local mode
4 sided support			
Modal mass (kg)	681	0.0	0.0

Fig. 5.4. Effect of boundary conditions on vibration modes of a 5m by 5m floor with three beams

The complexity of mode shapes for the illustrated cases and others (e.g. large floor system discussed in Chapter 4) makes it difficult to make intuitive judgments about how alterations in

construction features will affect modal responses, especially high order ones, of glulam beam-and-deck floors.

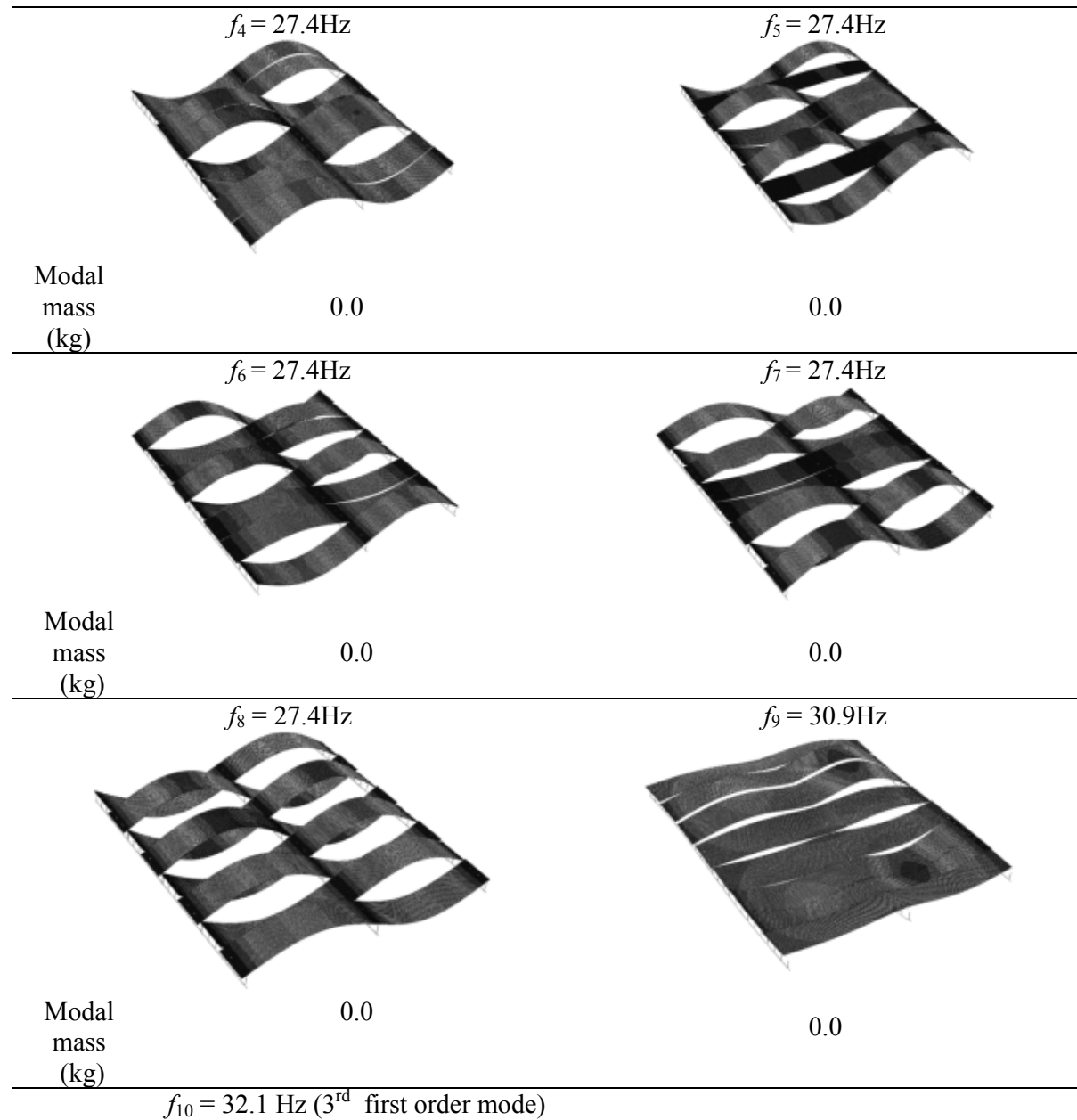




Fig. 5.5. Effect of boundary conditions on vibration modes of a 5m by 5m floor with three beams

All contemporary proposed vibration serviceability design criteria and associated analysis methods are predicated on consideration of motions associated with only the fundamental mode, or that and other global modes. For example, Eurocode 5 (CEN 2004) considers components of motion of first order global modes with frequencies less than 40 Hz. In consequence such criteria and analyses cannot realistically be expected to apply to situations like beam-and-deck floors where local modes can produce significant components of aggregated motions building occupants can experience under normal building occupancy conditions. Conversely this can be taken as a reason to require glulam beam-and-deck floors to be designed and constructed in ways that suppress local modes to a level that will not be detectable by building occupants (e.g. require deck elements to act monolithically by jointing them and installing adequate topping overlays). These considerations are symptomatic of why the author has refrained from trying to propose a new design criterion/criteria for glulam beam-and-deck floors. Instead, for the moment at least, he prefers to contribute to careful collection of information that supports creation of holistic concepts, and design and analysis methods applicable to a wide variety of lightweight floor systems.

5.2.3 Effect of number of beams

Previous discussion has addressed only floors with an odd number of evenly spaced beams, here consideration is extended to floors with an even number of beams. Fig. 5.6 shows mode shapes and modal frequencies for the three lowest order mode of 5 m by 5 m floors having between three or four beams. As can be seen the general natures of mode shapes were not affected by having four instead of three beams. The modal frequencies were not proportional to the number of beams despite all the shown modes being first order ones. This is consistent with experimental and numerical studies of Chapters 2 and 3. Again the explanation lies in the influential nature of beam and deck stiffnesses on details of the mode shapes. This reinforces other comments that effects constructions variables have on modal characteristics are not intuitively guessable except for the fundamental mode.

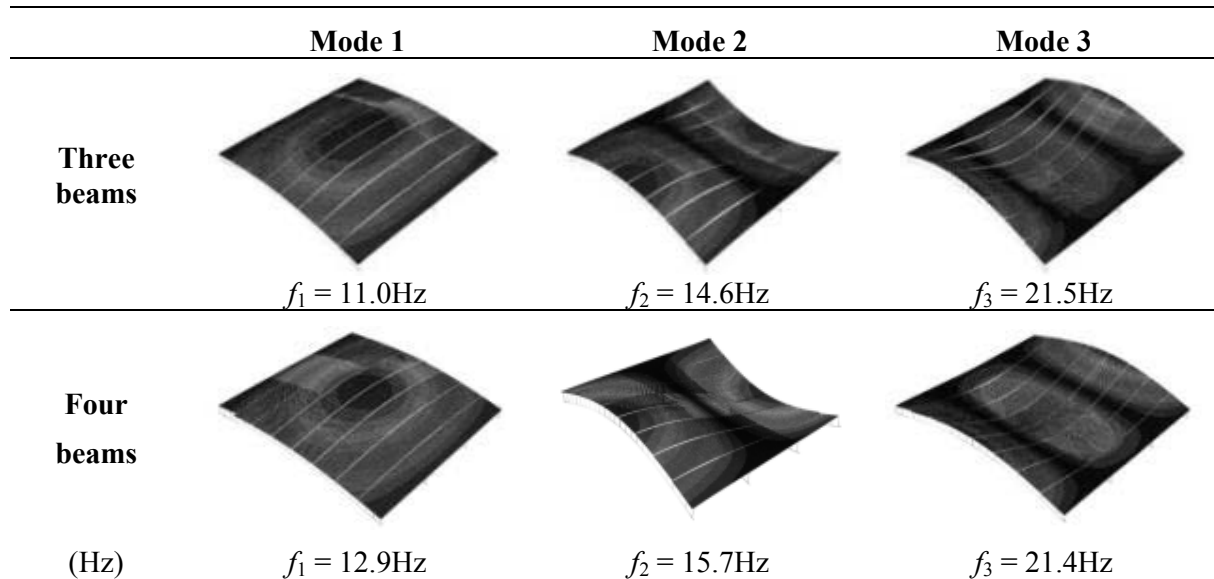


Fig. 5.6. Effect of number of beams on low order modes of a 5m by 5m floor

5.2.4 Prediction of deck deflection

An advantage of a calibrated FE model like that described in Chapter 3 is ability to reliably predict deflection of deck element at different points of the floor surface. Table 5.3 shows calculated displacements caused by a 1 kN concentrated gravity force placed at different locations on Floor 0. Selected force application locations are Points 1 to 5 in Fig. 5.7, and Points 6 to 10 are selected mirror displacement response locations. Importance of this capability relates to assessing performances of floors where maximum displacement would not occur at the center floor position (e.g. non-rectangular floors) or static displacement serviceability predictions.

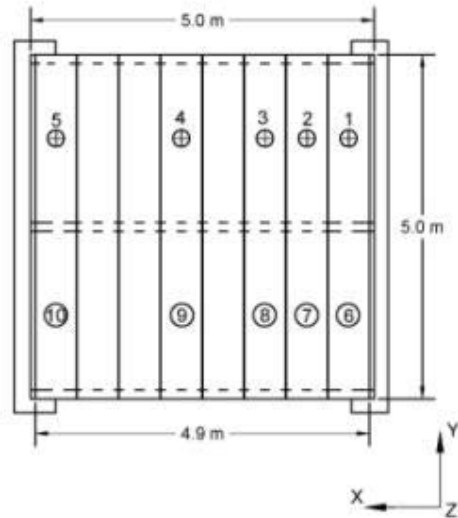


Fig. 5.7. Locations for application of a 1kN force on Floor 0

Table 5.3. Displacements caused by 1kN force on Floor 0

Load point	1	2	3	4	5
Displacement at load point (mm)	0.97*	1.1	1.29	1.29	0.97
Displacement at mirror measuring point (mm)	- 0.29** (6)***	-0.20 (7)	-0.11 (8)	-0.11 (9)	-0.29 (10)

* Positive indicates downward movement

** Negative indicates upward movement

*** Mirror measuring point

5.2.5 Number of first order modes up to 40Hz

Here n_{40} , the number of first order modes with natural frequencies up to 40 Hz, is calculated for glulam beam-and-deck floors, Table 5.4. The values are calculated using the FE models presented in Chapter 3. This follows the example of Eurocode 5 (CEN 2004) that bases estimates of peak velocity resulting from a unit impulse force on n_{40} . In effect this equates to filtering out contributions modes with frequencies $\geq 40\text{Hz}$ make to aggregated motions at floor surfaces when unit impulsive force is applied. Scope of Eurocode 5 provisions is only rectangular joisted floors with all edges simply supported, and there is no guarantee 40 Hz would be a suitable filtering frequency for other types of floor system. Therefore n_{40} values here are simply an example of the type of parameter that may be required for prediction of motions of beam-and-deck systems using a truly dynamic response based design method. Table 5.4 also shows n_{40} values calculated using the Eurocode 5 formula. The primary point to be derived from the tabulated values is that the number of

modes requiring consideration in estimation of design level floor motions is not constant, even for floors from the same classification. A second point of note is the unreliability of attempting to apply a calculation method (e.g. Eurocode 5) for n_{40} intended for one type of floor to others. Results in Fig. 5.4 indicate, for beam-and-deck floors where deck elements do not act monolithically that it cannot be supposed differences between FE analysis and Eurocode 5 estimates of n_{40} are simply the result of supporting only ends of beams versus simply supporting all edges of the floor. These comments are evidence of the complexity of how lightweight floors vibrate, and the inadvisability of trying to extrapolate design practices from one case to another without a thorough foundation of understanding.

Table 5.4. Predicted number of first order modes with frequencies up to 40 Hz

Floor	0	1	2	3
FE model (2 sided support)	4	4	3	3
Eurocode 5 formula	$2.47 \approx 3^*$	$1.98 \approx 2$	$1.44 \approx 2$	$1.40 \approx 2$

* Values rounded up to the nearest integer.

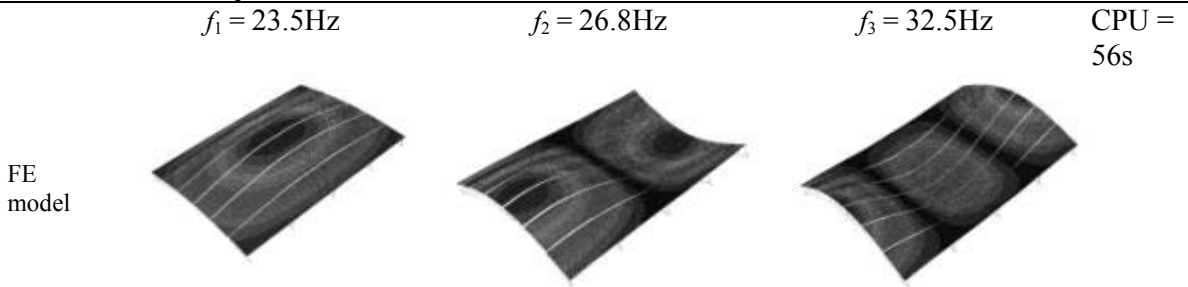
5.3 Further discussion points

5.3.1 Grillage models

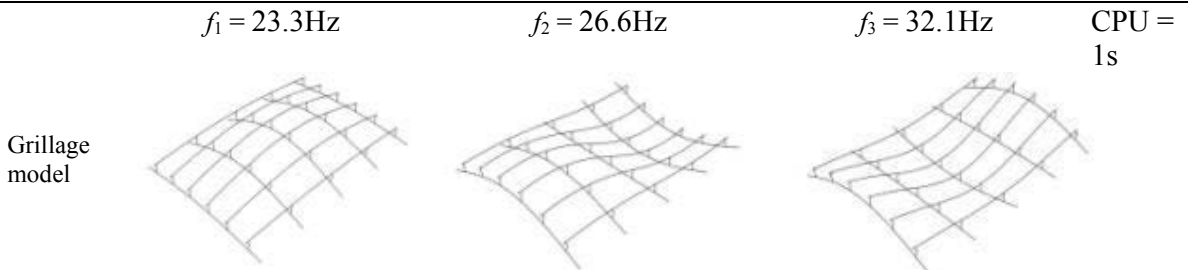
Ohlsson (1982) used a grillage analysis model to predict modal and time-history responses of joisted timber floors in the work that underpins already discussed Eurocode 5 provisions (CEN 2004). He modeled sheathing materials attached to tops of joists as equivalent beam elements.

Effects of torsion and rotary inertia were neglected in his model. Ohlsson reported excellent agreement between test and predicted responses of floor systems. The objective of grillage analyses presented here is to assess applicability of grillage analyses to prediction of modal responses of glulam beam-and-deck floors. Fig 5.8 compares outputs from FE models based on the approach in Chapters 3 and 4 with those produced by grillage analyses that replicate Ohlsson's approach. Those comparisons suggest grillage representations will be adequate for design level analyses of beam-and-deck floors.

A. Floor 3: 3.8m by 5m with 5 beams

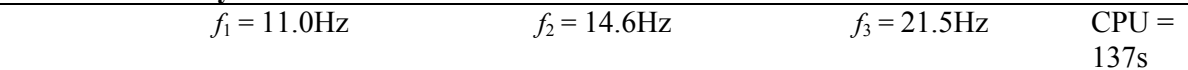


Total number of beam and shell elements 28901

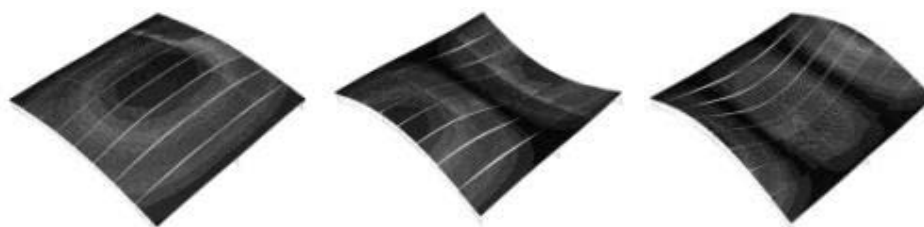


Total number of beam elements 880

B. Floor 0: 5m by 5m with three beams



FE
model



Total number of beam and shell elements 39865

$f_1 = 11.0\text{Hz}$

14.5Hz

21.2Hz

CPU =
31s

Grillage
model



Total number of beam elements 576

C. Large multi-bay floor (Chapter 4)

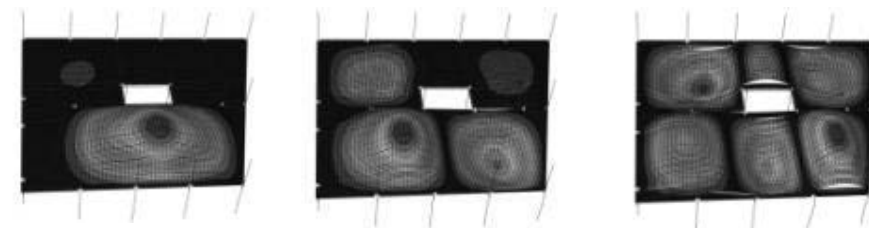
$f_1 = 7.4\text{Hz}$

$f_2 = 8.2\text{Hz}$

$f_3 = 9.2\text{Hz}$

CPU =
34s

FE
model



Total number of beam and shell elements 10666

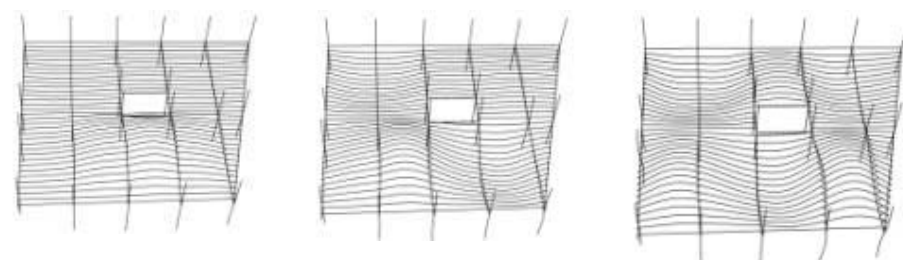
$f_1 = 7.7\text{Hz}$

$f_2 = 8.4\text{Hz}$

$f_3 = 9.2\text{Hz}$

CPU =
18s

Floor as
Grillage
Model

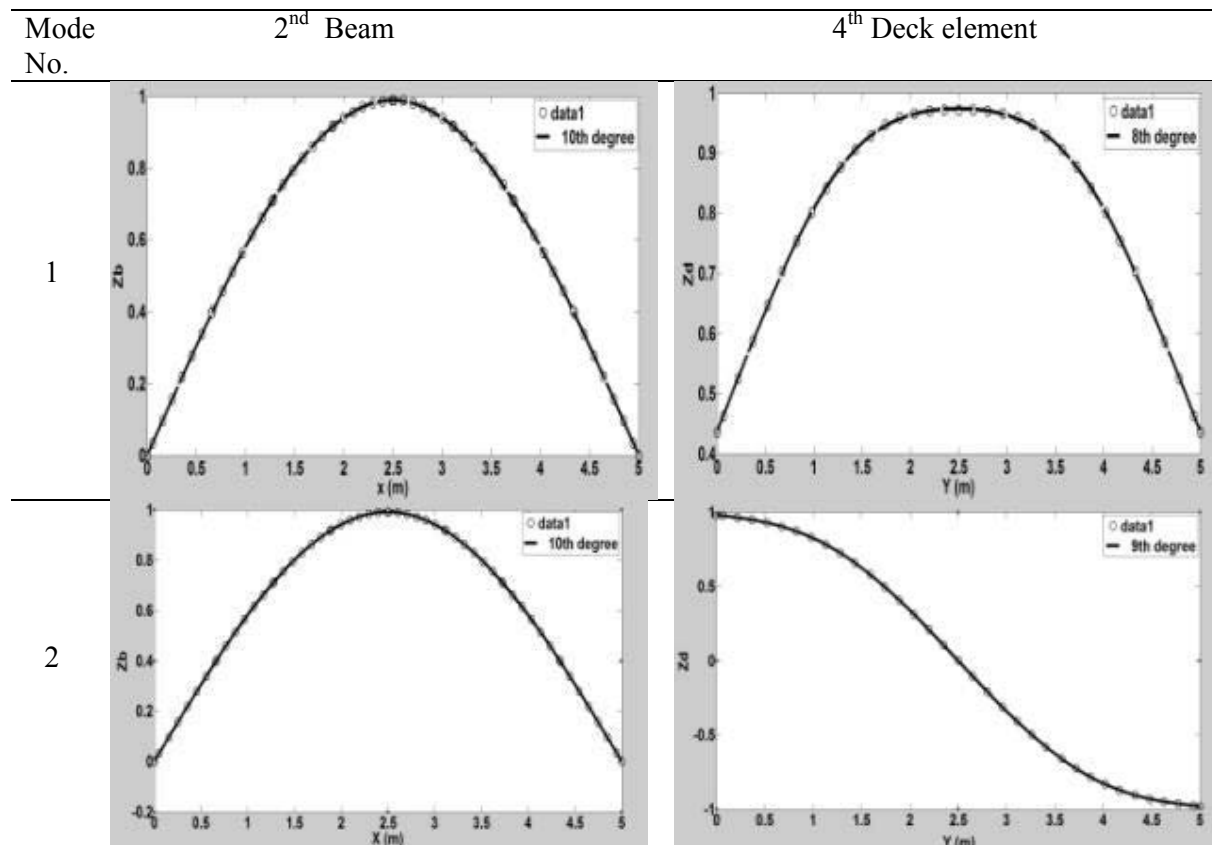


Total number of beam elements 4272

Fig. 5.8. Comparison between shell and grillage model predictions

5.3.2 Rayleighmethod

Approximate displacement polynomial and trigonometric functions can be used to imitate mass normalized mode shapes obtained from FE or grillage analysis models and substituted in Rayleigh equations to obtain approximate estimates of modal frequencies (Humar, 2012). Fig 5.9 gives two examples of polynomial functions fitted to FE model results. Parametric studies showed higher order polynomials yield better results than trigonometric functions. The approximated shape functions were applied in Maple software 18 (MapleSoft 2014) to predict modal frequencies. Normalized fundamental mode shape functions obtained from FE model outputs are given in Table 5.5 for Floor 0. Table 5.6 illustrates accuracy of Rayleigh method predictions for 2nd and 3rd modes of Floor 0.



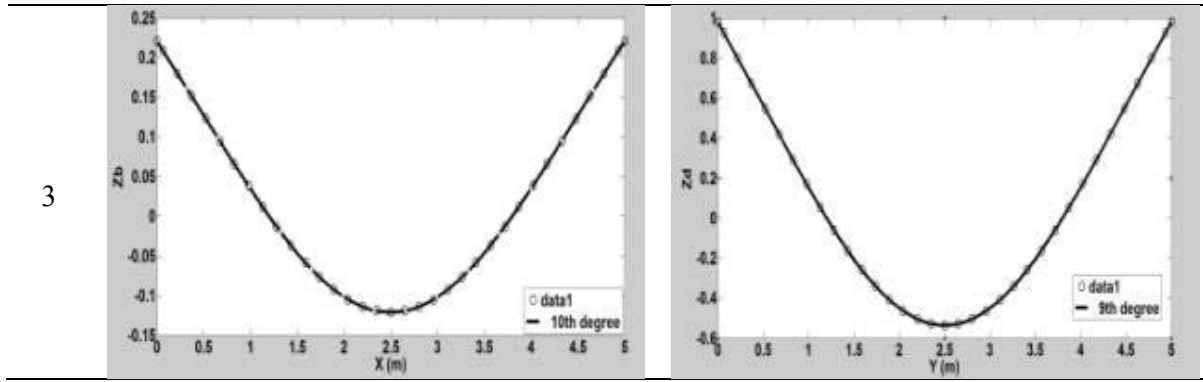


Fig. 5.9. Polynomial curves fits to obtain normalized mode shapes from FE model results

Table 5.5. Normalized mode shape equations

Prediction of fundamental frequency		
beam or deck (Mode shapes)	Polynomial function	Trigonometric function
Beam 1	$Zb = -3.8648e-06*x^{10} + 9.7351e-05*x^9 + 0.0010443*x^8 + 0.0062158*x^7 - 0.02243*x^6 + 0.050444*x^5 - 0.067523*x^4 + 0.031367*x^3 - 0.017399*x^2 + 0.29662*x - 5.8244e-05$	$Zb = 0.0008475 + -0.001007*\cos(x*0.6291) + 0.4692*\sin(x*0.6291)$
Beam 2	$Zb = 3.3801e-06*x^{10} + -8.3041e-05*x^9 + 0.00086966*x^8 + -0.005073*x^7 + 0.018015*x^6 + -0.039659*x^5 + 0.057544*x^4 + -0.091561*x^3 + 0.026186*x^2 + 0.61442*x + 0.00015323$	$Zb = 0.002515 + -0.002673*\cos(x*0.6294) + 0.986*\sin(x*0.6294)$
Beam 3	$Zb = -3.8648e-06*x^{10} + 9.7351e-05*x^9 + 0.0010443*x^8 + 0.0062158*x^7 - 0.02243*x^6 + 0.050444*x^5 + -0.067523*x^4 + 0.031367*x^3 + -0.017399*x^2 + 0.29662*x - 5.8244e-05$	$Zb = 0.0008475 + -0.001007*\cos(x*0.6291) + 0.4692*\sin(x*0.6291)$
Deck element 1	$Zd = -4.9085e-05*x^8 + 0.00098228*x^7 + -0.0077029*x^6 + 0.029563*x^5 + -0.055801*x^4 + 0.042995*x^3$	$Zd = 0.1501 + -0.05447*\cos(x*0.9079) + 0.06483*\sin(x*0.9079) + 0.002666*\cos(2*x*0.9079) +$

	$+ -0.022975*x^2 + 0.096699*x + 0.097949$	$0.01522 * \sin(2*x*0.9079)$
Deck element 2	$Zd = -0.00013445*x^8 + 0.0026904*x^7 + -0.021131*x^6 + 0.081487*x^5 + -0.15574*x^4 + 0.12512*x^3 + -0.067862*x^2 + 0.2522*x + 0.25351$	$Zd = 0.3892 + -0.1411*\cos(x*0.9098) + 0.1663*\sin(x*0.9098) + 0.006545*\cos(2*x*0.9098) + 0.0396*\sin(2*x*0.9098)$
Deck element 3	$Zd = -0.00019108*x^8 + 0.0038237*x^7 + -0.029997*x^6 + 0.11527*x^5 + -0.21838*x^4 + 0.17101*x^3 + -0.093205*x^2 + 0.37004*x + 0.37159$	$Zd = 0.5693 + -0.2066*\cos(x*0.9073) + 0.2467*\sin(x*0.9073) + 0.01044*\cos(2*x*0.9073) + 0.05853*\sin(2*x*0.9073)$
Deck element 4	$Zd = -0.00023272*x^8 + 0.0046567*x^7 + -0.036604*x^6 + 0.14146*x^5 + -0.27197*x^4 + 0.22295*x^3 + -0.12476*x^2 + 0.43711*x + 0.43542$	$Zd = 0.6665 + -0.2416*\cos(x*0.9062) + 0.2901*\sin(x*0.9062) + 0.01252*\cos(2*x*0.9062) + 0.06811*\sin(2*x*0.9062)$
Deck element 5	$Zd = -0.00022452*x^8 + 0.0044928*x^7 + -0.03526*x^6 + 0.13566*x^5 + -0.25778*x^4 + 0.20353*x^3 + -0.11104*x^2 + 0.43299*x + 0.43557$	$Zd = 0.667 + -0.2418*\cos(x*0.9072) + 0.2888*\sin(x*0.9072) + 0.01217*\cos(2*x*0.9072) + 0.06812*\sin(2*x*0.9072)$
Deck element 6	$Zd = -0.00019108*x^8 + 0.0038237*x^7 + -0.029997*x^6 + 0.11527*x^5 + -0.21838*x^4 + 0.17101*x^3 + -0.093205*x^2 + 0.37004*x + 0.37159$	$Zd = 0.5693 + -0.2066*\cos(x*0.9073) + 0.2467*\sin(x*0.9073) + 0.01044*\cos(2*x*0.9073) + 0.05853*\sin(2*x*0.9073)$
Deck element 7	$Zd = -0.00013878*x^8 + 0.0027769*x^7 + -0.021827*x^6 + 0.084349*x^5 + -0.1621*x^4 + 0.13249*x^3 + -0.071888*x^2 + 0.25301*x + 0.25349$	$Zd = 0.3899 + -0.1414*\cos(x*0.9118) + 0.165*\sin(x*0.9118) + 0.006162*\cos(2*x*0.9118) + 0.03975*\sin(2*x*0.9118)$
Deck element 8	$Zd = -4.9085e-05*x^8 + 0.00098228*x^7 + -0.0077029*x^6 + 0.029563*x^5 + -0.055801*x^4 + 0.042995*x^3 + -0.022975*x^2 + 0.096699*x + 0.097949$	$Zd = 0.1501 + -0.05447*\cos(x*0.9079) + 0.06483*\sin(x*0.9079) + 0.002666*\cos(2*x*0.9079) + 0.01522*\sin(2*x*0.9079)$
Raleigh method	11.09Hz	11.13Hz
FE model	11.03Hz	

Table 5.6. Comparison between FE model and Rayleigh method for the 2nd and 3rd modes

Using Polynomial function	FE model	Rayleigh method
---------------------------	----------	-----------------

Mode 2	14.6Hz	14.8Hz
Mode 3	21.5Hz	21.0Hz

As might be expected, in the Rayleigh method predicted modal frequencies agreed well with FE model predictions, demonstrating the former is an acceptable approach for glulam beam-and-deck systems.

5.4 References

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6 CHAPTER 6

GENERAL CONCLUSIONS AND RECOMMENDATIONS

6.1 General conclusions

The conclusions below address behaviour and design of glulam beam-and-deck floors and lightweight floors in general.

Many types of lightweight floors contain construction features that complicate their dynamic behaviour to a level that invalidates applicability of simplified representations of their modal response characteristics and dynamic responses to impacts or other excitations. This includes but is not be restricted timber floors. Ignoring feature variables like floor layout, support conditions, selection of intra-floor construction details, installation of floor overlay/topping materials, and presence of supported objects will lead to wrong design level deduction about susceptibility of prospective floor constructions to performance problems. This means that literature and design guidelines that recommend use of simplified design criteria are inherently unable to be reliable solutions. However, this does not mean creation of generally reliable and practically implementable design criteria and analysis methods is an unachievable goal. It means instead that the route to the goal is via development of methods that predict motions that will occur at surfaces of particular types of floor systems under particular building occupancy conditions, and use of motion assessment criteria directly based on how humans perceive and assess such motions. Foundations for this lie in application of advanced numerical dynamic analysis methods like FE approaches, and use of International Organization for Standards or similar motion assessment criteria. The author believes modern design and construction of lightweight substructures and superstructure systems

for buildings depends on embracing rather than shunning use of modern design tools and advanced serviceability performance concepts.

Little attention has been paid to the importance of structural damping on whether a structural system's motions will be acceptable to building occupants. Plus there has been little challenging of the notion that building occupant sensation and assessment of motions relates primarily to whether their internal organs will resonate. The walking test observations for a large glulam beam-and-deck floor reported in Chapter 4 are a strong illustration that the level of damping in a structural system is a strong determinant on whether humans will perceive floor motions as acceptable or unacceptable. Those observations also illustrate that simplified design concepts like requiring a system's fundamental natural frequency be greater than a limit like 8Hz (to avoid resonance of human organs) are unreliable. Improved vibration serviceability design criteria will be ones that recognize the importance of effective damping present in structural systems and relate human perception of motions to motion intensities rather than just the minimum frequency content of motions.

Neglecting how components associated with vibration modes other than the fundamental mode tends to underestimate motions building occupants will experience. This is particularly important when low order or other modal frequencies are clustered. Lightweight floors can be very prone to modal clustering, depending on their construction features. As discussed in Chapter 5, glulam beam-and-deck and undoubtedly other types of floor systems can exhibit local mode motions involving vibration of deck elements between supporting beams. This implies floor deck layers should be made monolithic (e.g. interconnecting them using edge-to-edge joints and/or overlays).

Isolated bay representation of floor systems for the purposes of design, based on supporting beam or wall positions with or without consideration of span continuity, is reportedly common practice to simplify design. Whether this is reliable depends on many factors, but always strongly on whether structural members are deliberately continuous across supports, or made effectively continuous by addition of what are termed non-structural materials to floors. This raises the issue of horizontal vibration transmissions within and between floor substructures. Analyses in Chapters 4 and 5 demonstrate that dynamic responses of glulam beam-and-deck floors are sensitive to variations in the support flexibilities. Although those analyses did not accurately represent entire superstructure systems an implication of those analyses is that substantive vertical vibration transmissions will occur in multi-storey superstructures containing such floors. Improved vibration serviceability design criteria will be ones that recognize the importance of continuities between bays that define floor substructures or create contiguity between floor structures. Improved criteria will also be ones that address, and help designers avoid, vertical vibration transmission in multi-storey superstructure systems.

It is important to emphasize the interdependence between architectural and engineering design decisions, and construction practice implement of design concepts. The literature on vibration serviceability performance of lightweight floors is full with examples of inconsistent and inappropriate applications of concepts. For example, if the architect and engineer for a project intend ends of beams or edges of slabs to be vertically supported or unsupported, or to be free to rotate on-site other design or construction decisions should not alter that in practice (e.g. not strengthening elements intended to improve seismic performance) without proper consideration of the issues. As another example, reports of failed attempts to solve floor vibration serviceability performance by adding concrete toppings to floors are quite common. A common reason for such

remediation failures is the presumption that the modal characteristics of the floor itself were the source of problem when in fact it was flexibility of supporting beams or walls. The lesson in this is to take a holistic approach to control of initial or remedial design decisions and their implementation, because the literature suggests research investigations and design/remediation guidance has often focused on solving the wrong problem or the right problem in a wrong way. The same applies in many other instances of course, with misguided initial design and retrofit of seismic force resisting systems being particularly common example.

6.2 Recommendations

The detailed recommendations below are based on research reported in this thesis. High research priorities are:

- 1) Defining the number of modes or cutoff analysis frequency to be considered in calculation of oscillatory motions of glulam beam-and-deck floors is a high research priority. This applies to evaluation of test data collected by OMA or other methods, and analyses by FE or other methods predicting responses to defined floor surface impacts. Presently it is unclear whether, for example, the 40Hz value specified by Eurocode 5 for rectangular joisted timber floors having all edges simply supported applies to other situations.
- 2) Defining dynamic walking strategies to be used in field or laboratory assessments of floor systems, and those walking or other footfall impacts to be used in numerical analyses of floor motions. For maximum benefit this would relate test or analysis situations to defined building occupancy situations.
- 3) Create best practice guidelines for design and construction of glulam beam-and-deck floors.

- 4) Create code provisions based on truly dynamic response and motion assessment criteria that negate need for *ad-hoc* analytical or empirical approaches specific to certain types of lightweight floor systems.

Appendix A Derivation of equation (2.3)

Starting from orthotropic plate theory mid-span deflection at the center of one a way spanning floor plate is derived as shown below:

For an orthotropic plate infinitely long in the y axis direction the differential equation for small elastic out-of-plane deflection (w) according to Timoshenko and Woinowsky (1959) is:

$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = q \quad (\text{A.1})$$

$$D_x = \frac{EI_y}{b} \quad (\text{A.2})$$

$$H = 2D_{xy} \quad (\text{A.3})$$

$$D_{xy} = D'_{xy} + \frac{k}{2b} \quad (\text{A.4})$$

$$D_y = \frac{Ebt^3}{12(b-c+\alpha^3 c)} \quad (\text{A.5})$$

$$\alpha = \frac{t}{z} \quad (\text{A.6})$$

where D_x , D_y and D_{xy} are respectively longitudinal and transversal flexural and torsional rigidities of the orthotropic plate; D'_{xy} is the torsional rigidity of the slab without beams; b is spacing between beams; t is the thickness of deck elements; k is the torsional rigidity of one beam; c is the width of one beam; E is the longitudinal modulus of elasticity of the element; z is the total height of the floor system including beam depth and thickness of the slab; and q is applied uniformly distribute load on the floor system (Fig. A1).

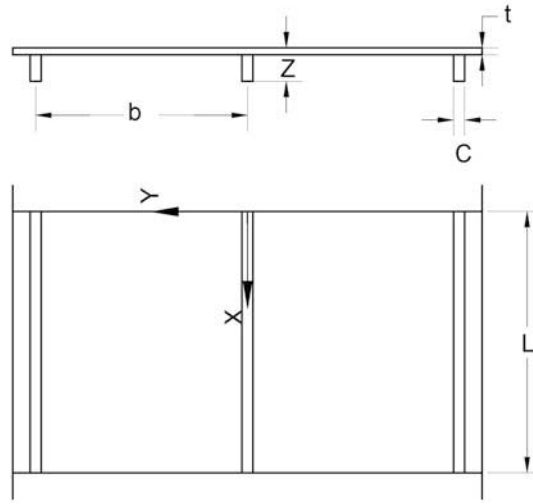


Fig. A1. Parameters used to define the equation

Neglecting shear deflections (Thompson et al. 1975) it is acceptable to assume that $H^2 \ll D_x D_y$, which leads to deflection of the plate under a concentrated load P , applied at the position $(x=\xi, y=0)$ being:

$$d_p = \frac{PL}{2\pi^2 \sqrt{D_x D_y}} \sum_{m=1}^{\infty} \frac{1}{m^3} (\alpha' \sin \frac{m\pi y}{\alpha'} + \beta' \cos \frac{m\pi y}{\alpha'}) e^{-m\pi y / \beta'} \sin \frac{m\pi \xi}{L} \sin \frac{m\pi x}{L} \quad (\text{A.7})$$

$$\alpha' = \frac{L\lambda}{\pi} \sqrt{\frac{2}{1-\mu}} \quad (\text{A.8})$$

$$\beta' = \frac{L\lambda}{\pi} \sqrt{\frac{2}{1+\mu}} \quad (\text{A.9})$$

$$\lambda = \sqrt[4]{\frac{D_{-y}}{D_{-x}}} \quad (\text{A.10})$$

$$\mu = \frac{H}{\sqrt{D_{-x}D_{-y}}} \quad (\text{A.11})$$

Displacement at the center of a floor plate corresponds to $(x=L/2, y=0)$, resulting in:

$$d_{P\text{-center}} = \frac{PL}{2\pi^2 \sqrt{D_{-x}D_{-y}}} \sum_{n=1}^{\infty} \frac{1}{n^3} \beta' \left(\sin^2 \frac{n\pi}{2} \right) \quad (\text{A.12})$$

or

$$d_{P\text{-center}} = \frac{PL}{2\pi^2 \sqrt{D_{-x}D_{-y}}} \sum_{n=1,3,5,\dots}^{\infty} \frac{1}{n^3} \beta' \quad (\text{A.13})$$

When $H^2 \ll D_{-x}D_{-y}$ the value of β' can be simplified to $\sqrt{2}L\lambda/\pi$. Writing equation (A13) terms for $n=1, 3, 5, \dots$:

$$d_{P-center} = \frac{\sqrt{2}PL^2(1.0516)\lambda}{2\pi^3\sqrt{D_xD_y}} \quad (\text{A.14})$$

Simplifying:

$$d_{P-center} = \frac{0.024PL^2}{D_x^{3/4}D_y^{1/4}} \quad (\text{A.15})$$

Finally, substituting $P = 1$ kN yields:

$$d_1 = \frac{0.024L^2}{D_x^{3/4}D_y^{1/4}} \quad \begin{array}{l} \text{Equation} \\ (2.3) \end{array}$$

Appendix B Joint push out test

Lateral load-slip responses of Heco-Topix Flange head screws were measured via push-out tests performed according to ASTM D1761–12 (ASTM 2012). This permitted estimation of suitable stiffness for link-element used to represent screw joints within FE models discussed in Section 3. Push-out test specimens consisted of 80mm x 130mm x 200mm deck elements and 304mm x 130mm x 130mm beam elements interconnected by 160mm Heco-Topix Flange head screws. Joint initial stiffness was estimated from 40 % of the peak load of load-slip curve (Fig. B1). The average stiffness per screw was determined to be 0.54×10^6 N/m to use in FE model.

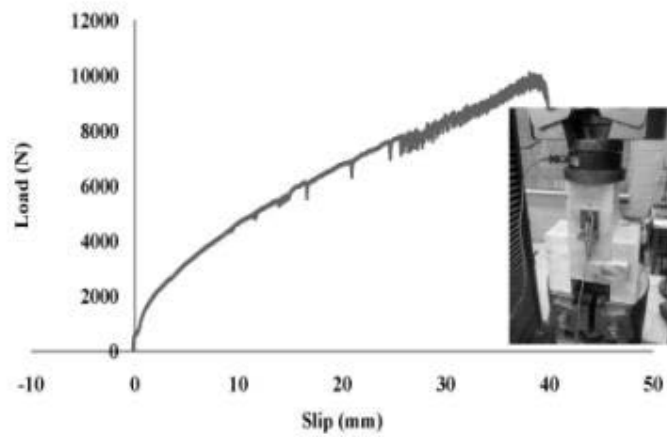


Fig. B1. Typical Loud slip curve for applied screw