

U.S. Mass Timber Floor Vibration

DESIGN GUIDE



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1

Introduction

1.1 Preface

The question of whether to use mass timber or competing materials for a building's flooring system often comes down to economics. In many floor applications, sizes of the mass timber panels and supporting framing, which significantly influence construction cost, are largely determined by limiting the floor vibrations perceived by occupants or sensitive equipment to acceptable levels. While vibration design is a primary driver of the framing system cost of floors, little information has been available to U.S. designers on how to design mass timber floors for vibration. Simple methods, such as the vibration span limit recommendations in the U.S. CLT Handbook (FPInnovations, 2013), can be appropriate for addressing vibration performance; however, these approaches have limited ranges of applicability.

The field of footfall-induced floor vibration performance and engineering design has a long history of research and design approaches. However, the guidance available was either developed for alternative materials and framing systems or follows European standards, with the inherent challenge of being based on non-U.S. design methods and the metric system of units. This design guide bridges the information and experience gap by synthesizing current design procedures and recommendations for mass timber floors and presenting the results in a format that is both approachable and useful to the U.S. engineering design community.

1.2 Scope

The scope of this guide focuses on the design of mass timber floor systems to limit human-induced vibration. The primary performance goal is to help designers achieve a low probability of adverse comment regarding floor vibrations in a manner consistent with the vibration design guides for steel and concrete systems. This includes excitation primarily from human walking as observed by other people in the building. Some treatment of design for sensitive equipment in response to human walking is also discussed. This design guide covers the range of currently available mass timber panels, including cross-laminated timber (CLT) manufactured from either solid sawn or structural composite lumber (SCL) laminations, nail-laminated timber (NLT), dowel-laminated timber (DLT) and glue-laminated timber (GLT), as well as their support framework of timber beams.

While many of the concepts and methods can be applied to other dynamic load sources, such as rhythmic excitations from exercise and machine-induced vibrations, these conditions are not covered in this guide. Other topics not covered include building code requirements, such as deflection limits and structural capacity needed to resist gravity or lateral loads, fire-resistance ratings and acoustics.

1.3 Target User

The target user of this guide is a design professional with working knowledge of mass timber structural design and some background knowledge of structural dynamics as related to floor vibrations. It may be particularly useful to design engineers with limited experience with vibration analysis, experienced multi-material engineers familiar with vibration analysis but unfamiliar with mass timber vibration, and applications engineers assisting manufacturers in the development of solutions and proposals for projects.

1.4 Notation

A	Cross-sectional area of member, in. ²
A_{gross}	Gross area of mass timber panel, in. ² or in ² /ft of panel width
a	Acceleration response of vibration, in./s ² or % g
a_1, a_2	Distance from the neutral axis of a composite to the neutral axis of a component, in.
a_{RMS}	Root mean square of acceleration of vibration, measured over a set period, in./s ² or % g
a_h	Peak acceleration for resonant response to harmonic h , in./s ²
$a_{h,\text{fw}}$	Peak acceleration for resonant response harmonic h which has been frequency-weighted for human sensitivity, in./s ²
a_p	Total peak acceleration for resonant response, in./s ²
$a_{p,\text{fw}}$	Total peak acceleration for resonant response which has been frequency-weighted for human sensitivity, in./s ²
$a_{\text{real},h}, a_{\text{imag},h}$	Components of the peak acceleration of all considered modes responding to harmonic h , in./s ²
$a_{\text{real},h,m}, a_{\text{imag},h,m}$	Components of the peak acceleration of mode m responding to harmonic h , in./s ²
b	Width of piece of lumber, layer, or strip of panel, in.
B_{eff}	Effective width of the floor deck when calculating the properties of a composite section, in.
d	Thickness or depth of a piece of lumber, layer or strip of panel, in.
E	Modulus of elasticity for timber, psi
EA_1, EA_2	Axial stiffness of a component of a composite assembly, lbf or lbf/ft of width
EA_{eff}	Effective axial stiffness of mass timber panel or composite section, lbf or lbf/ft of panel width
E_c	Modulus of elasticity of concrete, psi
$E_{c,\text{dyn}}$	Dynamic modulus of elasticity of concrete, psi
E_{eff}	Effective flatwise (out-of-plane) modulus of elasticity of a mass timber panel or component, psi
EI_1, EI_2	Bending stiffness of a component of a composite assembly, lbf-in. ² or lbf-in. ² /ft of width
El_{app}	Apparent flatwise (out-of-plane) bending stiffness of a mass timber panel including approximate of shear deflections, lbf-in. ² or lbf-in. ² /ft of panel width
$El_{\text{eff}}, El_{\text{eff},f}$	Effective flatwise (out-of-plane) bending stiffness of a mass timber panel or composite section, lbf-in. ² or lbf-in. ² /ft of panel width
E_{perp}	Modulus of elasticity of timber perpendicular to grain, psi
E_{ref}	Reference modulus of elasticity used in material definition in finite element analysis (FEA), psi
f'_c	Compressive strength of concrete, psi
f	Frequency of vibration, Hz
f_{11}, f_{22}, f_{12}	Modification factors to convert isotropic plate properties to orthotropic plate properties in some FEA methods (Section 5.4)
f_h	Harmonic frequency of loading, harmonic number times walking frequency, $h \cdot f_w$, Hz
f_m	Natural frequency of vibration of mode m , Hz
f_n	Natural or fundamental frequency of vibration; the lowest modal frequency of vibration, Hz
f_w	Walking frequency, Hz
F_h	Harmonic force for resonant analysis, lbf
g	Acceleration due to gravity, commonly taken as 386.1 in./s ²
G	Specified specific gravity of lumber, used for connection design and commonly based on dry-weight of wood normalized by the weight of water, unitless
GA_{eff}	Effective shear stiffness of mass timber panel or composite section, lbf or lbf/ft of panel width
$GA_{\text{eff},e}$	Effective shear stiffness in edgewise (in-plane) loading of mass timber panel or composite, lbf or lbf/ft of panel width

$G_{A,eff,f}$	Effective shear stiffness in flatwise (out-of-plane) loading of mass timber panel, lbf or lbf/ft of panel width
$G_{c,dyn}$	Dynamic shear modulus of concrete, psi
G_e	In-plane (edgewise, shear-through-the-thickness) shear modulus of mass timber panel or component, psi
G_{eff}	Effective flatwise (out of plane) shear modulus of a mass timber panel or component, psi
G_{ref}	Reference shear modulus used in FEA for mass timber panel, psi
G_t	Shear modulus of timber parallel to grain, psi
$G_{t,perp}$	Shear modulus of timber perpendicular to grain, i.e., rolling shear stiffness, psi
h	Harmonic number; an integer not less than 1 used to find harmonic frequencies of loading frequency, unitless
h_1, h_2	Thickness of a component in a composite assembly, in.
I	Moment of inertia for bending of mass timber panel or component, in. ⁴ or in. ⁴ /ft of panel width
I_{eff}	Effective moment of inertia for flatwise (out-of-plane) bending of mass timber panel or component, in. ⁴ or in. ⁴ /ft of panel width
$I_{eff,m}$	Effective impulse magnitude for mode m in transient response analysis, lbf-s
I_{gross}	Gross moment of inertia for flatwise (out-of-plane) bending of mass timber panel or composite, in. ⁴ or in. ⁴ /ft of panel width
l	Stride length of the walker, ft
L_{lim}	CLT Handbook method recommended span limit, ft
L	Span length of a structural component or bay, ft or in.
\bar{m}	Area or lineal mass on a component, (lbf s ² /in.)/ft ² or (lbf s ² /in.)/ft
m	Mode number; an integer not less than 1 used to identify the vibration mode under consideration
m_{11}, m_{22}, m_{12}	Modification factors to convert isotropic plate properties to orthotropic plate properties in some FEA methods (Section 5.4)
\hat{m}_m	Modal mass for mode m , lbf-s ² /in.
N_h	Number of loading cycles in sub-resonant correct factor
P	Assumed static weight of a walker, lbf
r	Distance between the neutral axes of two components of the composite assembly, in.
R	Response factor; a normalized measure of vibration response where the magnitude of vibration is divided by a standardized limit of human perception, unitless
t_p	Floor panel thickness, in.
T_n	Natural period of vibration, $1/f_n$, sec
T_w	Period of walking between steps, $1/f_w$, sec
v	Velocity response of vibration, micro-in./s
$v(t)$	Total velocity response as a function of time, micro-in./s
$v_{\frac{1}{3}}(t)$	Velocity response as a function of time for modes within one-third octave band around center frequency, micro-in./s
v_{13}, v_{23}	Modification factors to convert isotropic plate properties to orthotropic plate properties in some FEA methods (Section 5.4)
\hat{v}_m	Initial velocity from impulse loading of mode m for transient response, micro-in./s
$v_m(t)$	Velocity response from impulse loading as a function time for mode m , micro-in./s
v_{RMS}	Root-mean-square magnitude of the total velocity, measured over a set period, micro-in./s
$v_{RMS,\frac{1}{3}}$	Root-mean-square magnitude of the velocity, $v_{\frac{1}{3}}(t)$, micro-in./s

$V_{RMS, fw}$	Root-mean-square magnitude of the total velocity, which has been frequency-weighted for human sensitivity, micro-in./s
w	Area or lineal weight on a component, lbf/ft ² or lbf/ft
\bar{z}	Location of neutral axis of a composite section, measured from the bottom of the composite section, in.
\bar{z}_1, \bar{z}_2	Location of neutral axis of a component of a composite, measured from the bottom of the composite section, in.
α_h	Harmonic coefficient of loading for resonant analysis, also called dynamic load factors, unitless
$\gamma, \gamma_1, \gamma_2$	Partial composite action factor for a component of a composite; a value of 0.0 means no composite action, and a value of 1.0 means perfect composite action with no shear slip occurring between the component and the neutral axis of the composite, unitless
Δ	Deflection of a floor or component, in.
ζ	Damping ratio as portion of critical damping value, unitless
ζ_m	Damping ratio for mode m , unitless
$\mu_{e,m}$	Normalized modal amplitude of mode m at location of excitation, unitless
$\mu_{r,m}$	Normalized modal amplitude of mode m at location of receiver (observation), unitless
ρ	Density of a material, lbf/ft ³
$\bar{\rho}$	Specific gravity of a material, e.g., density of the material normalized by the density of water, unitless
$\rho_{h,m}$	Sub-resonant response reduction factor for limited walking distance for a harmonic and mode, unitless

Additional Subscripts

() ₀	Mass timber panel or composite assembly property in the major strength direction
() ₉₀	Mass timber panel or composite assembly property in the minor strength direction

1.5 Vibration Characteristics of Floor Structures

Vibrations occur to some degree in any floor system; however, some floor systems are more prone to vibration problems than others. For mass timber floors, designing for vibration can often limit the floor span.

To appreciate the similarities and differences between mass timber floors and those made from other materials, it is important to understand the key factors that influence floor vibration: mass, stiffness, span length and damping. It is generally favorable for vibration for a floor to have a high mass, high stiffness, short span and high damping. A summary of typical properties of common building framing materials is given in Table 1-1.

TABLE 1-1: Characteristics of popular structural materials

Material	Floor Dead Load (psf)	Damping (% of critical)	Material Stiffness (10 ⁶ psi)	Example Design Guidance*
Concrete	100-150	1-5	3.2-5.8	CRSI Design Guide 10, CCIP-016
Steel	50-100	0.5-5	30	AISC Design Guide 11, SCI P354
Mass timber	15-65	1-5	0.9-1.9	CLT Handbook
Light wood-frame	10-40	2-12	0.9-1.9	CSA O86-14, Annex A

* See *Useful Vibration Guidelines and References* below.

Based on the system properties of a given floor, certain materials and building typologies will be inherently more or less susceptible to vibration. Typically, shorter span and higher mass floors (e.g., some concrete slab floors) are less prone to vibration and often governed by strength or deflection. Conversely, longer span and lighter weight floors (e.g., some steel-frame floor systems) are more prone to vibration and more often governed by vibration performance criteria.

Because mass timber floors, including post-and-beam framing, have a high strength-to-weight ratio, they are strong and stiff enough to span relatively long distances. However, their lower weight also means that the design of mass timber floors tends to be controlled by vibration rather than strength.

The amount of guidance available for each material also influences how easily designs can be carried out. For example, while some steel systems are known to have vibration issues, there is a considerable amount of experience in the design of these floor systems. AISC Design Guide 11 (AISC, 2016) explicitly addresses the vibration design of steel floors and is widely used in the U.S. As mass timber is a relatively new innovation in the construction industry, broad guidance regarding vibration has been largely unavailable.

1.6 U.S. Building Code Requirements

The International Building Code (IBC) has no requirements for floor vibration performance. The deflection limits for floor members in IBC Section 1604.3 allow for a maximum deflection of $L/360$ under live loading and $L/240$ under combined dead plus live loading. For many floor framing systems and spans, simply complying with these requirements can result in floors that, while code compliant, are perceived as low quality and “bouncy.” The philosophy behind the IBC’s structural design requirements is that serviceability considerations above these minimums are beyond the scope of minimizing hazard to life and safety.

Despite this lack of prescriptive code requirements, careful selection and application of appropriate vibration performance targets in consultation with the rest of the project team is advisable. These performance targets can have a significant impact on the perceived quality of a building as well as the construction cost. As such, floor vibration performance is an area worthy of thoughtful deliberation and investment into education and execution.

1.7 Useful Vibration Guidelines and References

In developing this design guide, the following guidelines and references were found to be informative. For additional materials, please refer to the references at the end of this guide.

AISC Design Guide 11, 2nd Edition – Vibrations of Steel-Framed Structural Systems Due to Human Activity (AISC, 2016) – Developed for steel-framed structures and commonly used in the U.S. for all framing materials

CCIP-016 – A Design Guide for Footfall Induced Vibration of Structures (The Concrete Centre, 2007) – Originally developed for concrete systems in the United Kingdom; influential on the practice of vibration analysis and design for many framing materials in Europe and elsewhere

ISO/TR 10137 – Bases for design of structures – Serviceability of buildings and walkways against vibration (ISO, 2007) – Provides framework of vibration criteria and baseline frequency-weighted curves of limits of perception

ISO/TR 21136 – Timber Structures – Vibration Performance Criteria for Timber Floors (ISO, 2017) – Provides performance criteria for human acceptance of footfall-induced floor vibrations; intended for use with mass timber, heavy timber and timber joist floors

U.S. CLT Handbook (FPIInnovations, 2013) and *Canadian CLT Handbook* (FPIInnovations, 2019) – Both include a chapter on floor vibration developed by researchers at FPIInnovations

Additional documents of value include:

- *CRSI Design Guide 10 – Design Guide for Vibrations of Reinforced Concrete Floor Systems* (CRSI, 2014)
- *CSA O86-14 (Update No. 1) – Engineering design in wood. Annex A: A.8.5.3 Vibration performance of CLT floors* (CSA, 2016)

- JRC Scientific and Technical Report, EUR 24084, *Design of Floor Structures for Human Induced Vibrations*, EN (Feldmann et al., 2009)
- *Development of a Floor Vibration Design Guide Method for Eurocode 5* (Abeysekera et al., 2018)
- Eurocode 5 – EN1995: Design of Timber Structures (ECS, 2004)
- *Floor Vibrations – New Results* (Hamm et al., 2010)
- *Human Induced Vibration of Steel Structures (Hivoss) – Vibration Design of Floors Guideline* (Feldmann et al., 2008)
- National Building Code of Canada (NBCC) Structural Commentaries (User’s Guide – NBC 2015: Part 4 of Division B), Commentary D (NRC, 2017)
- *SCI P354 – Design of Floors for Vibration: A New Approach* (SCI, 2009)

1.8 Note to the User

As with any lightweight, long-span floor system, vibration performance may control a mass timber floor’s design up to and including panel selection (grade and thickness) and/or supporting member sizes and arrangement. To help designers assess the vibration performance of these types of floors, this guide suggests analysis approaches and performance target ranges.

The proposed performance targets for both acceleration and velocity represent a coarse indication of performance based on available data, not strict pass/fail acceptance criteria. These targets should result in a low (but not zero) probability of adverse comment; floors that exceed these targets slightly may not result in a significant increase in adverse comment. Research has shown that vibration response 2x larger than the targets proposed may result in adverse comment, but smaller changes are unlikely to be noticeable to a user (BS 6472, 2008; CCIP-016, 2007).

For any project, it is important to review vibration expectations with the client, understanding that vibration perceptions and expectations vary among individuals. The selection of vibration performance targets often involves a qualitative risk/cost assessment; reducing the predicted level of vibration by half generally requires a significantly more expensive floor structure.

For floor systems other than simple, wall-supported floors, until calibrated simplified methods capable of handling the spectrum of common floor design options are available, a modal response analysis with Finite Element Analysis (FEA) is generally recommended. FEA is a commonly used numerical method for solving differential equations. FEA is well suited to solving complex structural analysis problems and is implemented in commercially available structural analysis programs.

Reviewing the sensitivity to different modeling assumptions and varied excitation parameters can be useful when assessing the floor’s response. It is important to note that, regardless of the level of detail and precision in the analysis technique (e.g., modal response or even time history analysis), the results will remain a fairly coarse indication of a floor’s performance and will require the designer’s professional judgment. Whenever possible, FEA models of components, assemblies or systems should be calibrated to laboratory or in situ testing of similar elements.

2

Understanding Floor Vibration

While a complete discussion of structural dynamics is beyond the scope of this guide, this chapter provides background that may be useful to someone designing mass timber floors for vibration. It includes an introduction to the theory of dynamic problems and how they are analyzed, common measures of vibration and the merits of each, a discussion of how vibration is experienced by human beings, and an overview of approaches used to evaluate vibration performance.

2.1 Vibration Background

Vibration is generally characterized as the oscillating, dynamic (i.e., time-varying) response of a system to excitation. This response is highly dependent on the properties of the system and the excitation. Methods for calculating this response are presented in the following subsections.

2.1.1 Single-Mode Vibration Response

The free vibration response of a structure occurs when an initial excitation (typically an imposed force or displacement) disturbs the system from an equilibrium state, and is then allowed to vibrate freely. In the ideal conditions of linear stiffness and no damping, this movement is sinusoidal and characterized by two parameters: the amplitude of the peak response (displacement, velocity or acceleration) and the amount of time between peaks—the natural period of vibration, T_n . The inverse of the natural period is the natural frequency of vibration ($f_n = 1/T_n$).

To illustrate undamped free vibration, consider a single degree of freedom (SDOF) dynamic system with a mass (M) and spring with stiffness (K). If this system is acted on by an impulsive load or displacement, the system will oscillate around its equilibrium point (Figure 2-1).

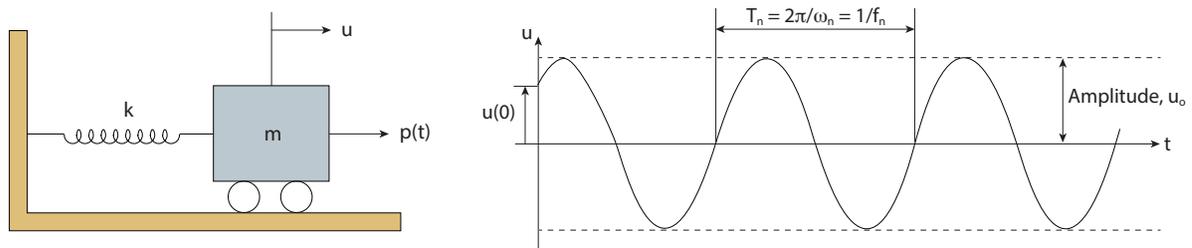


FIGURE 2-1: SDOF vibration

When damping is present, the free vibration will decrease with time. This decay in vibration over time is a result of energy loss from various sources and is generally accounted for with a damping ratio, ζ , measured as a percentage of the critical damping value. Critical damping is the lowest damping constant at which the system would return to its equilibrium state without oscillating.

Vibrations can be transient or resonant (steady state). The decaying, temporary response of the structure that occurs after a single loading event is referred to as a transient response. The steady-state, or resonant, response occurs when impulses are applied periodically to a system. If the system does not return to its equilibrium state before the new force occurs, the response to the new force will be added to the system, which may increase or decrease the overall response, depending on the phase of response for each force. For periodic loading, the response will eventually reach a steady state at the same frequency as the loading. The steady-state response depends on the relationship between the dynamic excitation, mass and stiffness, natural frequency, and damping of the system.

The response of a system can be calculated for many types of loading, including harmonic, periodic and random. Loads can be expressed as functions of force over time. Figure 2-2 shows the response of an SDOF system to periodic loading where the response is dominated by resonant or transient response.

For systems where the frequency of a periodic excitation, f_w , is close to the natural frequency, f_n , the amplitude of the response is extremely sensitive to the level of damping in the system. If the period of excitation of a repeating load is equal to the natural frequency, dynamic theory predicts infinite response at the SDOF system's natural frequency when no damping is present.

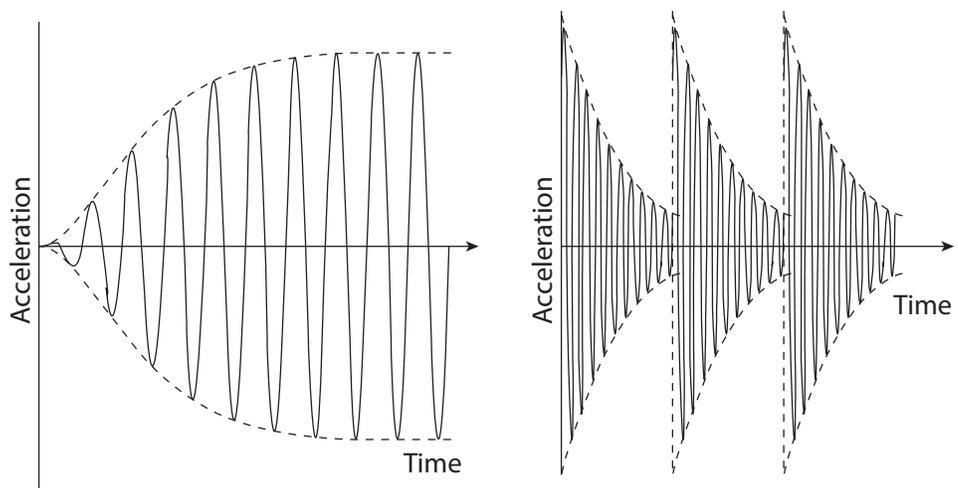


FIGURE 2-2: Build-up to resonant response (left) and transient response (right) of damped systems

2.1.2 Multi-Modal Vibration Response

For the free-vibrating SDOF system considered in 2.1.1, the amplitude of motion will depend on initial conditions, while the period between peaks, and natural frequency, will not. This fixed pattern of motion is referred to as a “mode of vibration” (mode) and can be described in terms of its natural frequency and deformed shape.

Real structures do not have one single degree of freedom and have many modes of vibration (Figure 2-3). For perfectly elastic vibration, each mode is mathematically a distinct SDOF system, with its own natural frequency, deformed shaped and effective mass, called the modal frequency, mode shape and modal mass, respectively. The lowest modal frequency of a system is termed the “fundamental frequency,” labeled f_1 . The sum of the responses from individual modes to an excitation provides the total vibration response of the multi-degree of freedom (MDOF) system through a process mathematically called modal superposition.

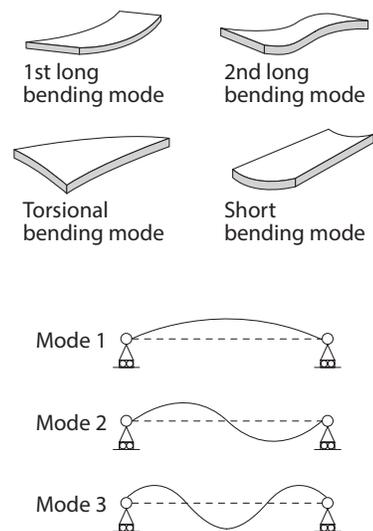


FIGURE 2-3: Vibration mode shapes

2.2 Structural Response to Footfall Forces

Footfall vibration is the dynamic movement induced into a floor by a person (or people) walking on that floor. This movement is generally not concerning from a structural safety perspective. However, it can interfere with the function of sensitive equipment or cause discomfort to building users. Beyond discomfort, discernment of floor vibrations can be a significant factor in the perceived quality of the floor system.

The load to the floor system from walking is a complicated function dependent on the walker weight and walking pace (measured in steps per second or Hz) and varies from person to person. The vibration response of the floor to this load will depend on factors that include the floor's stiffness, weight and damping properties.

To an observer near a person walking, two types of vibrations may be perceived.

1. Transient vibrations immediately following a single footfall
2. Resonant vibrations resulting from build-up to multiple footfalls

Whether resonant or transient vibration is the dominant phenomenon being experienced depends on the relation between the walking frequency and the fundamental frequency of the floor. Walking frequencies typically occur within the relatively narrow range of 1.6 to 2.1 Hz (Section 3.6.1), so the fundamental frequency of the floor is the primary determinant of which type of vibration dominates the floor response to walking. Extreme examples of fundamental frequencies of floors can range from below 3 Hz to over 20 Hz.

2.2.1 Low-Frequency Floors

Floors with fundamental frequencies at the lower end of the range are susceptible to the build-up of a resonant response to walking. When a resonant build-up occurs, the resonant response will tend to be more perceptible than any transient response that occurs.

Resonance can occur when the excitation frequency is close to the fundamental frequency or another significant modal frequency of the structure. Footfall excitation is composed of numerous excitation frequencies instead of a single frequency. In this case, resonance will occur when the significant modal frequencies of the structure are harmonic with one or more of the excitation frequencies. The harmonic frequencies of an excitation frequency are the integer multiples of the walking frequency. For example, if $f_w = 2$ Hz, the second, third and fourth harmonics of the walking frequency are at 4, 6 and 8 Hz, respectively. A floor with a natural frequency of 4, 6 or 8 Hz can have resonant build-up of vibrations to a walking frequency of 2 Hz. The magnitude of resonance decreases at higher harmonics and, beyond the fourth harmonic of the walking frequency, the resonant build-up of a structure will generally be small and largely damped out between excitations.

Low-frequency floors are therefore those with a fundamental frequency, f_n , less than or equal to the fourth harmonic of the maximum anticipated walking frequency ($4f_w$). This leads to an upper limit for low-frequency floor behavior in the 8 to 10 Hz range based on the range of walking frequencies anticipated.

As the magnitude of resonance is limited by damping, the magnitude of the vibration response of low-frequency floors is highly dependent on the damping level.

2.2.2 High-Frequency Floors

Floors with fundamental frequencies above the fourth harmonic of the walking excitation (typically 8 to 10 Hz) have a response dominated by a transient response and are called high-frequency floors. The vibration caused by a single step dissipates between one step and the next due to the large number of vibration cycles between steps. The magnitude of the response of high-frequency floors is impacted by the level of damping in the system, but to a much smaller degree than in low-frequency floors.

2.3 Measuring Vibration

The displacement, velocity, or acceleration response can each be used to measure vibrations. In practice, acceleration and velocity are the most used quantities as they are relatively easy to measure in-situ with readily available instrumentation such as digital accelerometers, and they are more directly related to what occupants perceive.

When working with field measurements or response history analyses, it is convenient to define simplifying measurement quantities that tell us something about the user experience without having to inspect the entire signal (i.e., acceleration or velocity history).

The absolute maximum (peak) value is a frequently used simplifying measurement quantity. Judgments of performance based on peak values can sometimes be misleading since this value neglects subtleties such as response duration or decay, or can be a result of instantaneous events (e.g., caused by stepping immediately adjacent to a sensor) that would not be experienced by a typical occupant. Peak values are used in situations where the floor system is governed by its resonant response, since the effects of a single event or step will not be significant over the resonant build-up of multiple steps.

The root mean square (RMS) is an alternate, commonly used signal processing technique that better accounts for changes in amplitude over time and is frequently applied to both transient and resonant vibration problems. An RMS value is a weighted average of the vibration response over a sampling time period, and is defined as follows:

$$v_{\text{RMS}} = \sqrt{\frac{1}{T} \int_0^T v(t)^2 dt} \quad a_{\text{RMS}} = \sqrt{\frac{1}{T} \int_0^T a(t)^2 dt}$$

The RMS response of a time history will always be less than the peak value. For example, the RMS of a pure sinusoidal response is $1/\sqrt{2}$ (approximately 71%) times the peak response. The RMS response value is sensitive to the sampling period; for a signal with significant decay, a long sampling period will underestimate the vibration response if RMS is used. When performing physical instrumentation, ISO 10137 recommends measuring RMS response using a one-second sampling interval that occurs over the period of maximum vibration intensity.

2.4 Human Perception of Vibration

While vibration response can be experimentally measured, how humans respond to vibration is highly subjective and difficult to quantify. The main factors are the vibration's direction, magnitude (amplitude), frequency, duration, and the orientation and activity of the observer. Because floor vibration problems occur in the vertical direction, the values discussed in the rest of this guide are limited to vertical motion (normal to the floor plane). Human sensitivities to horizontal vibrations are important in some situations—e.g., when designing tall structures subject to wind-induced vibration and for long-span pedestrian bridges.

The amplitude of vibration has the largest and most straightforward impact on perception. The baseline RMS acceleration perceptible by humans is $0.0005g$ (0.193 in./s^2) commonly written as 0.05% g or 0.5 milli-g.

Humans cannot accurately differentiate between small changes in amplitude. As a rule of thumb, changes in the range of 10 to 20% are unlikely to be noticed; the vibration amplitude often has to be doubled before the change is perceived (CCIP-016).

Human sensitivity to vibration also depends on the frequency at which it occurs. Similar to light and sound, humans are more sensitive to vibration in certain frequency ranges. For example, an acceleration of 0.5% g at 6 Hz feels different to a person than 0.5% g at 20 Hz due in large part to our physiology. For vibration in the vertical direction (z-axis), motion is most detectable between the frequencies of 4 and 8 Hz, which corresponds to the frequencies people feel in their gut. To account for variability of sensitivity across these ranges, a frequency weighting function is applied to the baseline RMS acceleration, resulting in a piece-wise linear curve of constant vibration perception (Figure 2-4). Accelerations below the curve are generally not perceptible to humans.

Velocity is also commonly used to quantify floor vibration. Similar to acceleration, the human perception limit of velocity depends on its frequency. Perception sensitivity is assumed to be constant, with an RMS velocity of $4,000$ micro-in./s (or mips) above 8 Hz and to decrease proportionally with decreasing frequency below 8 Hz (AISC, 2016; CCIP-016). The piece-wise linear curve of constant vibration perception to RMS velocity in both standards is shown in Figure 2-5.

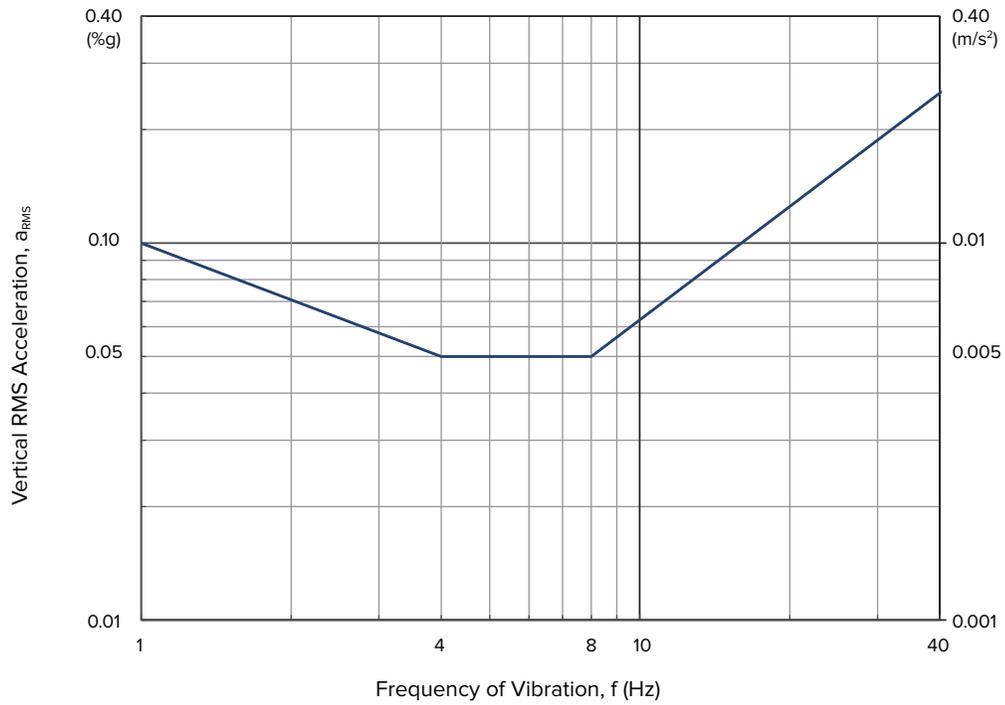


FIGURE 2-4: Human limits of perception of z axis RMS acceleration (ISO 10137)

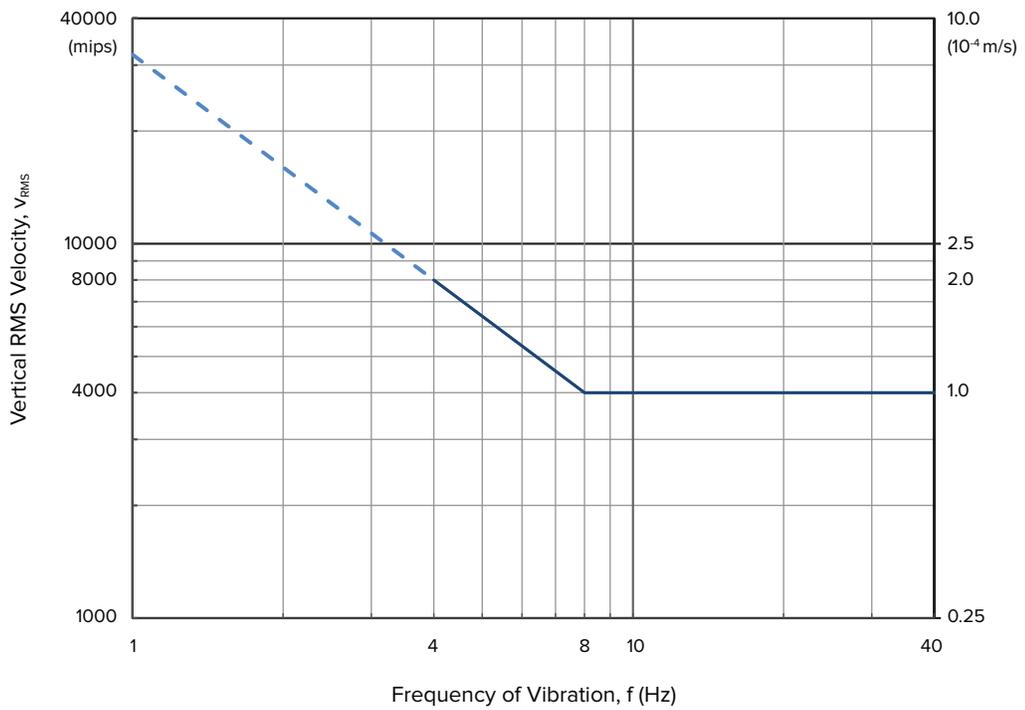


FIGURE 2-5: Human limits of perception of z axis RMS velocity

Figures 2-4 and 2-5 illustrate that acceleration and velocity each have a frequency range where human sensitivity is relatively constant (i.e., 4 to 8 Hz for acceleration and > 8 Hz for velocity). These frequency ranges tend to inform which metric is used to assess floor performance: acceleration is typically used to assess low-frequency floors ($f_n < 8$ Hz), and velocity is used for high-frequency floors ($f_n > 8$ Hz). This convention is convenient because it allows for the floor response to be compared to a performance limit in the form of a single number (e.g., 0.5% g or 4,000 micro-in./s). Where floor frequencies fall near the transition point of 8 Hz, frequency weighting can be applied to both acceleration and velocity limits to arrive at the appropriate limiting values. This is discussed further in Chapter 3.

The activity of the person perceiving the vibration also affects how an occupant both experiences and tolerates vibration. Generally, occupants are more tolerant of vibration when working in a busy workshop, or walking, than while sitting at a desk or trying to sleep. The anticipated occupant activities influence the selection of performance targets, as discussed further in Section 3.7.

Finally, the duration and number of vibration events also affects a user's tolerance. Short-term, infrequent events are likely to be more tolerated than frequent and/or long-lasting vibration events. The RMS acceptance criteria in the vibration perception curves (ISO 10137) do not account for duration and number of vibration events, which can result in conservative results for some scenarios. Some existing techniques do account for duration and frequency of vibration events; the most common is the vibration dose value (VDV). VDVs are not commonly used in practice and should be used only with sound engineering judgement. VDV criteria are useful where short-term events are not appropriately captured by RMS (ISO 10137; BS 6472).

2.5 Methods for Evaluating Vibration

Defining a precise pass/fail acceptance criterion guaranteeing 100% satisfaction of building occupants is impractical because of the variation in each person's tolerance for vibration and uncertainty in the excitation. There are a number of methods for evaluating floor vibration that intend to provide acceptable performance for most building occupants, ranging from "rules of thumb" to dynamic calculations evaluating floor vibration velocity or acceleration. Following are some of the most common methods for estimating a floor's performance as acceptable or not.

2.5.1 Deflection Limits

Although deflection is not a direct indicator of vibration, it is an indicator of the relative stiffness and mass of a floor and, therefore, closely related to the natural frequency of the floor system.

Some building codes include deflection limits to help limit vibration, such as the Eurocode. The IBC provides an L/360 deflection limit to the design level (not expected) live loads; however, this limit is not considered a good design rule for floor vibration, especially with longer spans. A target developed for light-frame floor systems is to limit the deflection under the design level live loads to L/480, or a maximum deflection of 1/2-in. (Woeste & Dolan, 1998; Tsuda & Woeste, 2015).

Deflection limits are valuable for the serviceability of floor framing systems. However, they are not sufficient to address vibration of mass timber floors, given the variety of potential framing arrangements, assemblies and occupancies.

2.5.2 Fundamental Frequency Limits

The fundamental frequency of a floor is a significant contributor to a floor's vibration performance. As described in Section 2.2, the fundamental frequency can be used to categorize floors into low-frequency floors responding with resonance and higher-frequency floors responding as transient.

Floors with a fundamental frequency in the range of walking frequencies (1.6 to 2.1 Hz) can have significant resonant response. Additionally, such floors can be subject to intentional resonant excitation to the detriment of other occupants or structural integrity. Fundamental frequencies of the floor in the range of walking frequencies are highly discouraged. AISC Design Guide 11 recommends avoiding floors with $f_n < 3$ Hz and the NBCC suggests a minimum natural frequency of 4 Hz.

If the fundamental frequency is sufficiently high, occupant perception of floor vibration to human excitation will likely not be an issue. For example, Dolan et al. (1999) demonstrated that, for light wood-frame floor systems, keeping the fundamental frequency of floors above 14 Hz for occupied floors generally results in vibration acceptable to typical occupancies, such as residences and offices. Similarly, AISC Design Guide 11 states that the “authors are not aware of any vibration serviceability problems, nor of any experimental data related to human comfort, for steel-framed floors with natural frequencies above 15 Hz.”

Floor performance between these limits (i.e., $3 \text{ Hz} < f_n < 15 \text{ Hz}$) is transitional and less predictable. It is in these cases that the more detailed methods discussed in following sections of the guide are essential to understanding floor behavior. It should be acknowledged that almost all practical mass timber floors fall within this frequency range.

2.5.3 Empirical Approaches

Some design methods are based on empirical data from experimental and analytical studies. Floor systems are evaluated by comparing their properties to relationships or simplified equations derived from this research. When used correctly, these methods simplify the vibration design process and can avoid the need for significant dynamic modeling. Following are two notable empirical methods for timber systems.

2.5.3.1 CLT Handbook Method

This method was developed by FPIinnovations based on laboratory tests and is applicable to simply-supported CLT panels on solid supports, e.g., bearing walls. Further discussion of this topic is provided in Section 4.2.2.

The CLT Handbook method is applicable to CLT floor panels supported on bearing walls with a fundamental frequency not less than 9 Hz. This method is explicitly not applicable to low-frequency floors.

2.5.3.2 Hamm-Richter Method

In Hamm et al. (2010), an empirical approach was presented for timber floors based on laboratory and in-situ observations. For office and multi-unit residential applications, this method recommends that the short-term static deflection due to a 2 kN (450 lbf) point load be no greater than 0.5 mm (0.0197 in.). For floors with a fundamental frequency between 4.5 Hz and 8 Hz, more in-depth dynamic modeling is recommended to maintain a maximum acceleration limit of 0.5% g. Floors below 4.5 Hz are not recommended.

2.5.4 Dynamic Model-Based Methods

The floor vibration design methods in AISC Design Guide 11, CCIP-016, SCI P354 and other documents use dynamic models of floor systems to predict the acceleration or velocity response to walking excitations. The magnitude of the predicted response is then compared to appropriate acceptance criteria. The possible dynamic analyses range in sophistication from approximate hand calculations to FEA of 3D models used with post-processing tools to combine the effects of many modes of vibration.

As this guide builds significantly on these sources, summaries of the methods in both AISC Design Guide 11 and CCIP-016 are provided as related to this guide.

2.5.4.1 AISC Design Guide 11, 2nd Edition, Vibration Methods

The objectives of AISC Design Guide 11, 2nd Edition, are to “provide basic principles and simple analytical tools to evaluate vibration serviceability of steel-framed structural systems to human activity.” Although it was developed for steel-framed floors, the guide’s methods are general and applicable to other materials. This has allowed it to become a popular vibration design guide for all materials in the U.S. Chapter 2 describes evaluation criteria for human comfort, and Section 2.2 pertains to walking excitations of floor systems.

Section 2.2.1 presents a performance acceptance method based on an estimate of the peak acceleration valid for low-frequency floors—defined as floors with a fundamental frequency of 9 Hz or lower. Using a range of walking frequencies from 1.6 to 2.2 Hz and an assumed walker weight of 157 lbs, a simplified prediction of the peak floor acceleration is calculated in Equation 2-6 using only the fundamental natural frequency, associated modal weight (mass) of the floor, and assumed damping. For offices, residences and other quiet spaces, the estimated peak acceleration is to be kept not greater than 0.5% g.

Section 2.2.2 of AISC Design Guide 11 presents a performance acceptance method for human comfort for high-frequency floors ($f_n = 9$ to 15 Hz) that is similar to the method for low-frequency floors. While considering a transient floor response to walking, an equivalent sinusoidal peak acceleration (ESPA) is calculated using the fundamental natural frequency, walking frequency, modal weight, and damping. The predicted peak acceleration is the ESPA and is compared with a target acceleration criterion. Because the fundamental frequency is at or above 9 Hz, the target acceleration increases with the frequency of the floor system following a multiplier of the baseline limit of perception as shown in Section 2.4 of this guide. Neither simplified method in AISC Design Guide 11 Chapter 2 is applicable to irregular framing or cantilevers, and they both include calibration factors based on experience and testing with steel-framed floors.

Chapter 3 of AISC Design Guide 11 provides methods for estimating the natural frequency of regular steel-framed floor systems using hand calculations. Many of the relationships in this chapter may be useful for approximate dynamic analysis of open-frame mass timber floor systems.

Chapter 4 of AISC Design Guide 11 provides methods for estimating the modal mass for low-frequency floors and examples of implementing the low-frequency floor method of Chapter 2.

Chapter 6 of AISC Design Guide 11 provides a method of designing floors for sensitive equipment and sensitive occupancies with more stringent requirements than typical occupancies. However, despite that stated scope, it is not uncommon for structural engineers familiar with this method to use the generic velocity limits of AISC Design Guide 11 Section 6.1.4 for typical office and residential occupancies. In this method, the designer selects a maximum considered walking frequency and a target RMS velocity performance, measured in micro-in./s.

Chapter 7 of AISC Design Guide 11 provides recommendations of FEA-based predictions of steel floor systems, which are useful background for modeling open-frame mass timber floor systems. This chapter also presents an evaluation method using modal analysis results from FEA for both low-frequency and high-frequency floors, using a frequency response function analysis of locations of interest in the floor framing.

2.5.4.2 **CCIP-016 Vibration Methods**

While a cement and concrete industry publication, this guide intends to be applicable to “any type of structure on which people walk” and “any form or construction material,” including timber. Given this broad scope, the following describes some of the included methods and their application.

Chapter 3 of CCIP-016 provides a good overview of vibration criteria from different sources, along with recommendations that are mostly consistent with AISC Design Guide 11.

Chapter 4 provides a vibration evaluation method dependent on dynamic modal analysis of the structure using a modal superposition method. Section 4.2 presents a procedure for low-frequency floors, which may exhibit resonance. The vibration metric for this procedure is a response factor, R , which is the estimated peak acceleration divided by the ISO baseline limit of perception of acceleration converted to a peak value. The baseline peak acceleration is 0.071% g in the 4 to 8 Hz range, which is consistent with the RMS baseline of 0.05% g in ISO 10137 (assuming sinusoidal response). Section 4.3 presents a procedure for the transient response in high-frequency floors, using detailed modal analysis results to predict an RMS velocity. The general methods for low-frequency floors and high-frequency floors rely on using FEA to determine the modes of vibration within the range relevant to vibration performance.

Section 4.4 of CCIP-016 provides simplified and approximate methods for regular rectangular floor plates supported on a line of closely spaced columns or load bearing walls. The approximations in this method account for varying stiffness between the primary and secondary strength directions of the floor plate and multi-span conditions. The simplifying approximations provided in this section were developed for concrete floor plates.

Chapter 5 of CCIP-016 provides worked examples, and Appendix A provides a presentation of dynamic properties of structural systems which may be useful to reference.

3

Vibration Design Considerations

This chapter gives an overview of the main properties that are important for vibration design, including recommendations specific to mass timber floor systems.

3.1 Floor Weight and Mass

The amount of mass assumed present for floor vibration analysis should represent an expected level in the occupied building and include anticipated permanent dead loads and some proportion of the live load. Such estimated loads may differ from those specified when designing for strength and deflection. In cases where the in-service floor weight is highly uncertain, or may vary depending on use, it may be prudent to perform a sensitivity analysis or examine multiple assumptions for floor weight. For traditional office-like construction, AISC Design Guide 11 recommends that 4 psf for normal mechanical and ceiling installations be added to the structural system as supplemental weight for estimating day-to-day dead loads. Because timber construction tends to be significantly different than traditional steel construction, a project-specific level of supplemental weight can be considered.

The live load to include is typically a small portion of that required by the building code for strength or deflection checks, but needs to account for the expected weight of furniture, equipment and contents on a typical day. Table 3-1 provides a summary of live loads recommended in AISC Design Guide 11 to serve as a basis for the project under consideration. Where a more refined value is not available, the expected live load is often taken as 10% of the code-specified live load.

Adding mass to the constructed assembly is a common approach for improving vibration performance for mass timber floors (e.g., concrete topping), but care must be taken to ensure the fundamental frequency of the floor is not reduced to frequencies near resonance, offsetting the benefit of additional mass.

TABLE 3-1: Recommended live load for vibration analysis

Occupancy	Approximate Expected Live Load (psf)
Paper office	11
Electronic office	6-8
Residence	6
Assembly area	0
Shopping mall	0

3.2 Damping

Damping accounts for the energy dissipation within a dynamic system. For floor vibration, energy will be lost through friction and dispersion to other structural or nonstructural elements or connections. Damping increases with the addition of finishes, furniture, etc. Table 3-2 provides guidance for typical damping ranges based on experimental data and contemporary research.

A sensitivity analysis is recommended where damping is found to significantly influence the predicted floor performance. Generally speaking, low-frequency resonant response is more sensitive to changes in damping than high-frequency floors. See Figure 3-6 for a comparison of resonant response with various damping ratios at a range of walking frequencies.

TABLE 3-2 Proposed mass timber floor damping ratios

Category	Range of Damping ζ (% critical)	Discussion
Lightly damped	1-2%	The lower end includes bare floors without topping and with minimal furnishing. The higher end includes floors with concrete topping and furnishings.
Moderately damped	2-4%	Lower values include bare timber-concrete composite floors, or timber floors with a floating concrete layer and full furnishings. The higher values include floors with floating floor layers, raised floors, full furnishings and mechanical systems. Floors with both furnishings and permanent partitions, not otherwise accounted for, could also be represented at the higher end of this damping range.
Heavily damped	4-5%	Floors in this range represent the upper limit of inherent damping. These floors likely include floating toppings, raised floors, suspended ceilings, furnishings, fixtures and/or permanent partitions not otherwise taken into account.
Explicit damping control	5%+	Generally, mass timber floors do not have more than 5% damping unless explicit damping control (e.g., a tuned mass damper) is added. These systems are beyond the scope of this guide.

3.3 Component Stiffness

Wood is an anisotropic material, meaning its properties vary by axis. For example, the elastic modulus of wood perpendicular to grain is considerably less than its parallel-to-grain value. The anisotropic behavior of wood is especially important to consider in the stiffness estimation for vibration evaluation. Furthermore, most mass timber structural members used in modern construction are laminated (i.e., built up from smaller pieces of wood). Established methods and assumptions for estimating wood properties are commonly based on the elastic modulus parallel to grain, which is a readily available property of wood.

Concrete and gypcrete are commonly included in the assembly of mass timber floors and can potentially contribute to the floor system’s stiffness. Therefore, their properties will be also discussed briefly.

3.3.1 Wood

The elastic modulus of wood parallel to grain, E , for the base wood used in a mass timber structural member, serves as the reference value for other wood stiffness properties. The NDS® Supplement – National Design Specification® Design Values for Wood Construction (AWC, 2018) provides tabulated values for E for solid sawn lumber. See Table 3-3 for typical assumed wood properties based on the elastic modulus.

3.3.2 Glue-Laminated Timber

Glue-laminated timber (glulam) beams, girders and columns can be represented with basic material properties E and G_t for simplified calculations per Table 3-3 and with reference to the NDS Supplement. When glulam members are placed in plank orientation to form a flat panel, the product is referred to as GLT. These panels can be represented similarly to NLT or DLT, as described in Section 3.3.4.

TABLE 3-3: Assumptions for wood stiffness properties (PRG 320-19)

Property	Value
Elastic modulus parallel to grain, E	E
Elastic modulus perpendicular to grain, E_{perp}	$E/30$
Shear modulus parallel to grain* G_t	$E/16$
Shear modulus perpendicular to grain, i.e., rolling shear, $G_{t,perp}$	$E/160$

* When shear deflection contribution is included explicitly, the shear-free elastic modulus can be approximated as 1.03 and 1.05 times E for dimension lumber and glulam, respectively (NDS, 2018, C.3.5.1)

3.3.3 Cross-Laminated Timber

The effective flatwise (out-of-plane) bending stiffness ($EI_{eff,f,0}$ and $EI_{eff,f,90}$) and shear stiffness ($GA_{eff,f,0}$ and $GA_{eff,f,90}$) for CLT grades and layouts are provided in manufacturers' literature and product evaluation reports. The section properties for basic grades and layouts can also be found in the ANSI/APA PRG 320: Standard for Performance-Rated Cross-Laminated Timber (PRG 320-19); however, engineers should consider which panels are available from manufacturers before using values in PRG 320-19 for design. A model to calculate design values for alternative grades or layouts of CLT can be found in Appendix X3 of PRG 320-19.

Detailed models of CLT explicitly consider shear deformations of the panels when subject to out-of-plane bending, which can be accomplished using a Timoshenko beam or thick plate models. For simplified calculations that do not explicitly consider shear deformations, an apparent stiffness value, EI_{app} , is used. Refer to Chapter 10 of the 2018 National Design Specification® (NDS®) for Wood Construction (AWC, 2018) for the calculation of this value. The shear contribution to overall deflection decreases with increasing slenderness, while the potential for vibration increases.

When considering the effects of partial or fully composite assemblies with CLT, the axial stiffness of the CLT is needed. Effective axial stiffness values are not currently provided in manufacturer data or PRG 320-19. However, the effective axial stiffness values of a CLT panel, $EA_{eff,f,0}$ and $EA_{eff,f,90}$, can be calculated according to:

$$EA_{eff,f,\theta} = \sum_{i=1}^N E_i b_i d_i$$

Where:

θ is panel orientation under consideration (0 for major strength direction, 90 for minor strength direction)

i = the layer number

N = number of layers in CLT

E_i = elastic modulus of layer i , either E or E_{perp} of layer, psi

b_i = width of layer i , 12 in. for properties of a 1-ft strip, or B_{eff} for flange over composite beam

d_i = thickness of layer i , in.

CLT can also be manufactured with one or more layers of structural composite lumber (SCL) instead of solid sawn lumber. One such product is the mass plywood panel (MPP), which uses 1-in. plywood laminations that are structurally qualified as laminated veneer lumber (LVL). For this type of CLT, the strength and stiffness properties are specified in the manufacturer's literature and product approval reports.

3.3.4 Nail- or Dowel-Laminated Timber and T&G Decking

NLT, DLT and tongue-and-groove (T&G) decking stiffness properties are simply the bare wood properties oriented appropriately. Where only simplified calculations are necessary for vibration evaluation, NLT, DLT and T&G decking can be considered as having only one-way spanning capability with $E_{eff,0}$ and $G_{eff,0}$ per Table 3-4. For more detailed modeling, the additional properties in Table 3-4 are required.

TABLE 3-4: Stiffness properties of NLT, DLT and T&G decking

Property	NLT	DLT	T&G Decking
$E_{eff,0}$	E	E	E
$I_{eff,0}$	NLT Guide*	Per manufacturer's specifications	I
$G_{eff,0}$	G_t	G_t	G_t
$E_{eff,90}, G_{eff,90}, G_e$	0**	0**	0**

* NLT panels with staggered or random layouts without finger-joined lumber have significant stiffness reductions. Refer to the NLT U.S. Design & Construction Guide (BSLC, 2017). Where continuous laminations are used, $I_{eff,0} = bd^3/12$.

**Use properties of topping layer where applicable.

3.3.5 Concrete/Gypcrete Topping

While equations for calculating the elastic modulus of concrete, E_c , can be found elsewhere (e.g., ACI 318-19 Building Code Requirements for Structural Concrete [ACI 2019], where $E_c = 57000\sqrt{f'_c}$ in U.S. customary units) and are used regularly when checking strength and deflection. An additional consideration when performing vibration assessment is the low-strain dynamic modulus, $E_{c,dyn}$, versus moderate-strain static elastic modulus of cementitious materials. It is recommended that the elastic modulus of concrete, E_c , be calculated by typical methods and amplified by a factor of 1.35 to account for low-strain and dynamic effects present in vibration loading. The shear modulus, G_c , can then be estimated as $G_c = 1.35 \cdot 0.4 \cdot E_c$. Due to the low stresses induced by vibration excitation, the concrete topping may be assumed to be uncracked for determining stiffness properties. Stiffness of gypcrete toppings is not as well understood and has been observed in some cases to be significantly lower than lightweight concretes of comparable strength. It is the authors' recommendation that gypcrete toppings be treated exclusively as additional mass to the assembly unless the stiffness is verified by the manufacturer or by other rational methods.

3.4 Composite System Effects

Mass timber floor panels are rarely, if ever, used as the only component of a floor assembly, and other structural and nonstructural components can increase the overall effective stiffness. When considering occupant perception of vibrations from walking, it is generally conservative to ignore any such stiffening. However, considering the stiffening effect can produce a more economical way to meet project performance targets.

As concrete is the most common topping layer used in the composite design of mass timber systems, such design is frequently called timber-concrete composite (TCC) design.

Two approaches can be taken. One is to consider composite behavior to meet the strength, long-term deflection, and vibration performance targets. Strength and deflection requirements are typically dictated by the building code and can be met with composite connection detailing. In this guide, such an approach is called **explicit composite behavior**.

A second approach considers an estimate of composite behavior in the vibration performance of a floor but not for the strength and long-term deflection performance of the floor system. This approach is called **incidental composite behavior**. As long as the non-composite system can meet the strength and deflection requirements, incidental composite behavior can take advantage of a full floor assembly to more efficiently meet vibration performance targets.

Table 3-5 shows a comparison of design approaches for explicit versus incidental composite behavior.

TABLE 3-5: Composite action used to meet performance targets

Composite Behavior	Strength	Deflection	Vibration
Explicit	Yes	Yes	Yes
Incidental	No	No	Yes

3.4.1 Composite System Stiffness

While a full treatment of design to achieve explicit composite behavior is beyond the scope of this guide, the following provides background theory in sufficient detail to implement incidental composite behavior for vibration considerations. For more information on composite system design, see sources such as the Design Guide for Timber-Concrete Composite Floors in Canada, published by FPInnovations (Ceurrer-Auclair, 2020)

Consider a composite system with two layers, component 1 and 2, as shown in Figure 3-1.

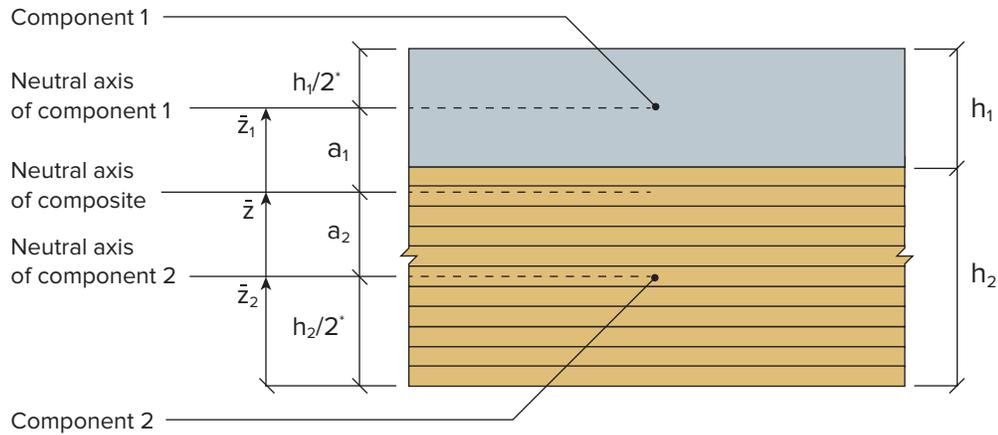


FIGURE 3-1: Two-component composite system

**h/2 only when centroid of component centered on depth of component*

The effective flexural stiffness (EI_{eff}) of the composite system can be calculated using the following equation based on the parallel axis theorem and incorporating partial composite action:

$$EI_{eff} = EI_1 + EI_2 + \gamma_1 * EA_1 * a_1^2 + \gamma_2 * EA_2 * a_2^2$$

Where:

EI_1 and EI_2 are flexural stiffness of components

EA_1 and EA_2 are axial stiffness of component

a_1 and a_2 are the distances from the neutral axes of the components to that of the composite

γ_1 and γ_2 are the partial composite action factors of the components

The partial composite action factors, γ_1 and γ_2 , each have values ranging from 0 to 1. A value of 0 represents no composite action, and a value of 1 represents full composite action. A value of 1 is used for the component in which the neutral axis of the composite section is located. For partial composite action, a value less than 1 is used for the component in which the neutral axis is not located.

The following notation is also used in composite beam theory:

r is the distance between the neutral axis of component 1 and component 2; $r = a_1 + a_2$

\bar{z} is the location of the neutral axis of the composite, typically measured from the bottom of component 2

\bar{z}_1 is the location of the neutral axis of the component 1

\bar{z}_2 is the location of the neutral axis of the component 2

Given properties of the components, and assumed values of γ_1 and γ_2 , the location of the neutral axis of the composite section can be calculated using any of the following:

$$a_1 = \frac{\gamma_2 * (EA)_2 * r}{\gamma_1 * (EA)_1 + \gamma_2 * (EA)_2}$$

$$a_2 = \frac{\gamma_1 * (EA)_1 * r}{\gamma_1 * (EA)_1 + \gamma_2 * (EA)_2}$$

$$\bar{z} = \frac{\gamma_1 * (EA)_1 * \bar{z}_1 + \gamma_2 * (EA)_2 * \bar{z}_2}{\gamma_1 * (EA)_1 + \gamma_2 * (EA)_2}$$

Where both components are symmetric about their individual neutral axes and are in contact with each other, $r = (h_1 + h_2)/2$.

After calculation of the neutral axis of the composite, the assumption that γ is less than 1 for the correct component (e.g., Component 1 versus Component 2) needs to be verified. For a thin upper layer, such as typical toppings over a mass timber panel deck, verify the assumption $\gamma_1 \leq 1.0$ by confirming $a_2 \leq h_2/2$. Similarly, for a thick upper component, verify the assumption $\gamma_2 \leq 1.0$ by confirming $a_1 \leq h_1/2$.

The effective shear stiffness of the composite system is simply the sum of each shear stiffness:

$$GA_{\text{eff}} = GA_1 + GA_2$$

3.4.2 Cementitious Toppings on Mass Timber Panels

One form of composite system design is to design for explicit composite behavior between an upper structural concrete topping layer and the lower mass timber panel floor deck (Figure 3-2). For the design of such a system, it may be justified to use a higher partial composite action factor for vibration analysis than the one used for strength and deflection calculation because connections are expected to behave more rigidly for lower loads. Selection of partial composite action factors for explicit composite design are beyond the scope of this guide and may be informed by testing of the detailed connection between the upper and lower components.

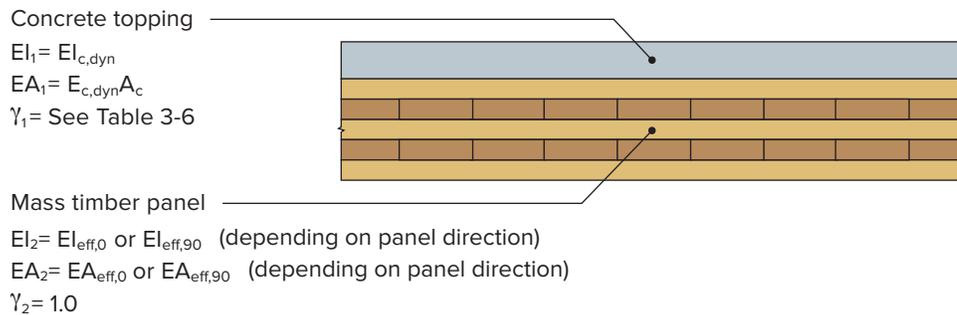


FIGURE 3-2: Thin concrete topping on mass timber panel

When concrete is cast directly on mass timber panels and a nominal positive connection between the topping layer and panels is provided, a degree of incidental composite behavior may be considered. An example of such a nominal connection is screws installed at 24 in. o.c. in each direction before the placement of the concrete topping. This nominal connection is not designed to transfer all of the shear forces, but it helps to maintain continuous contact between the topping and mass timber panel where the forces from vibration are partially resisted through surface friction. The combined section properties can be calculated using equations in Section 3.4.1. Ideally, the degree of composite behavior can be based on testing of a floor assembly similar to the floor under consideration in design.

The presence of an acoustic mat or slip-sheet between a mass timber panel and any cementitious topping layer reduces friction and the level of incidental composite action. When composite action is assumed not to occur, the contribution of the stiffness of the concrete topping can be considered for vibration analysis through simple summation:

$$EI_{\text{eff}} = EI_1 + EI_2$$

$$GA_{\text{eff}} = GA_1 + GA_2$$

This approach is similar to that for concrete on metal deck in AISC Design Guide 11 and timber research such as Hamm et al. (2010). Limited testing has shown that even with the presence of an acoustic mat, a small level of composite action can occur.

For a nonstructural cementitious topping layer, such as gypcrete, the material stiffness is challenging to quantify and control. If such properties can be adequately justified, incidental composite behavior similar to concrete might be possible; however, the suggestions provided for concrete toppings and mass timber panels are based only on observations of floor assemblies with concrete topping layers.

Where testing or other guidance is not available, limited testing performed by the authors and colleagues in the industry suggests partial composite action factors in ranges shown in Table 3-6. Engineering judgment should be exercised when selecting the appropriate γ for a specific design scenario. It is conservative to take the low end of the ranges.

TABLE 3-6: Suggested composite action between floor assembly components

Case		Partial Composite Action Factor (γ)	
		Strength & Deflection	Vibration Design
Concrete topping on mass timber panel detailed as a TCC system with explicit composite action		From testing or detailed analysis	Potentially higher than for strength & deflection
Incidental Composite Action	Concrete topping cast directly on mass timber floor with nominal connection	N/A ¹	0.15-0.50 ²
	Concrete topping cast directly on mass timber floor with no connection	N/A ¹	0.05-0.15 ²
	Concrete topping on acoustic mat or slip-sheet on mass timber panel	N/A ¹	0-0.05 ²
	Mass timber panel in direct contact with timber beam with clamping connection	N/A ³	0.5-1.0 ²

¹ Only the mass timber panel is considered; the cementitious topping layer is ignored.

² Values are based on limited testing and field observations.

³ Only the beam is considered; potential contribution from the mass timber panel and topping is ignored.

As timber-concrete composite design is not currently recognized in U.S. design codes and standards, a hybrid approach is also possible where explicit composite design is used for vibration only. When following this approach, the designer will calculate the partial composite action factor, γ , based on the stiffness provided by the connectors between the components of the composite system.

3.4.3 Mass Timber Panel on Supporting Beams

Explicit composite action between a glulam beam and a mass timber floor panel for strength and deflection is sometimes accomplished using screws, notches or other fastening hardware. Even when explicit design for composite action is not pursued for strength or deflections, incidental composite behavior may assist in the vibration response of a floor. Positive clamping between the panel and beam promotes the development of friction between the elements along their length as shown in Figure 3-3. For example, partially-threaded screws installed with the threaded portion only in the girder, spaced no more than 24-in. o.c., can be assumed to achieve some degree of composite action for vibration-level forces.

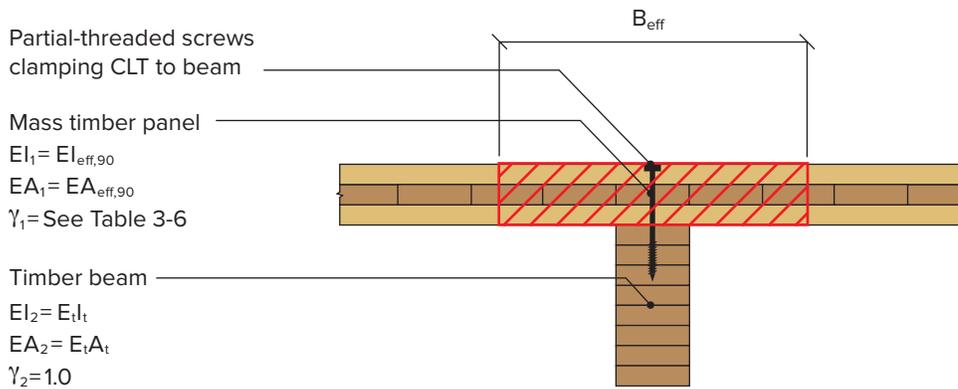


FIGURE 3-3: Mass timber panel on timber beam

The effective width of the compression flange, B_{eff} , used to determine the properties of the deck contributing to the combined section can be taken as the lesser of $L/2.5$ or the spacing between beams for T-sections and the lesser of $L/5$ or half the spacing between beams for L-sections. L is the span of the beam (AISC Design Guide 11).

When floor panels are supported on a system of purlins supported on girders, the same behavior can apply to the purlins and girders depending on the details. For a system with the panels supported on purlins with dropped girders as shown in Figure 3-4, the deck and the girder act independently without composite action. If the panel is directly supported by and clamped to the girder as shown in Figure 3-5, the deck and girder can act compositely. However, note that the top of the purlins may be designed and positioned slightly above the top of the girder to assist with constructability, which does not allow for direct contact and friction between the panel and girder.

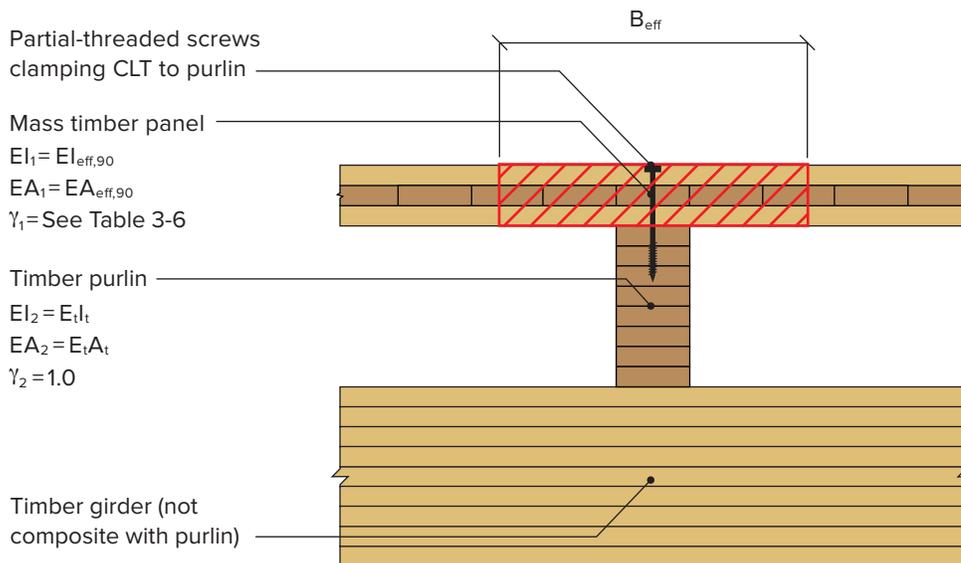


FIGURE 3-4: Mass timber panel on purlin bearing on girder

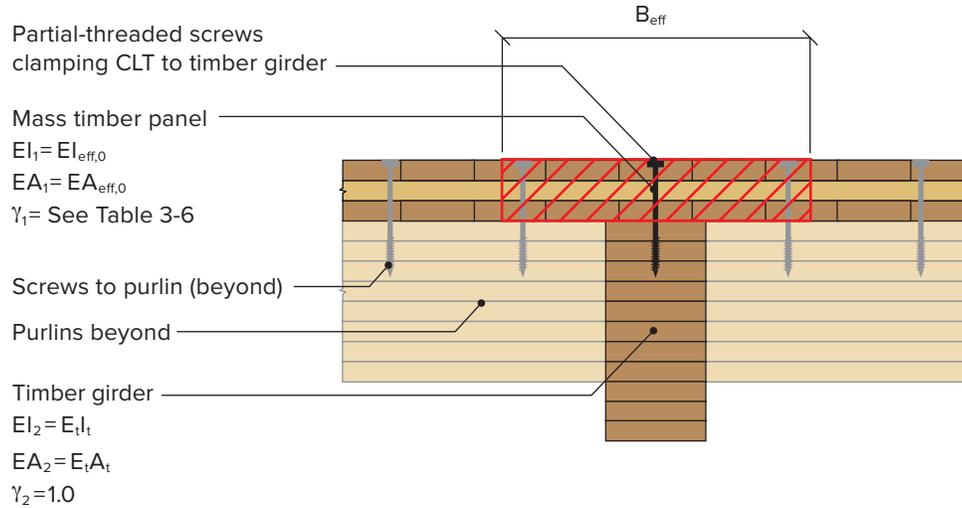


FIGURE 3-5: Mass timber panel on flush girder

3.5 Floor System Layout Considerations

Architectural layouts and framing approaches can have significant impacts on the vibration performance of a floor system.

3.5.1 Continuity and Isolation of Critical areas

Continuity over multiple spans and floor bays influences the vibration performance of a floor. A panel or beam spanning continuously over multiple spans has a higher mass mobilized during vibration (modal mass) relative to a similar component over a single span. This can decrease the magnitude of the vibration response, but it also ties spans together, sometimes negatively affecting occupants in adjacent spans. For example, someone walking in a corridor could affect someone sleeping on the other side of the wall. The benefit of reducing the magnitude of vibration should be weighed against the impact of transmitting vibration across boundaries where sensitivities can be increased.

The primary direction of continuity is along the length of the multi-span panel or beam. Panel products with two-way action, like CLT, can exhibit significant stiffness and continuity in the transverse (typically minor strength) direction of the panel. Connection details between panels can influence the mass mobilized in the transverse direction and should be accounted for. Concrete toppings can provide significant continuity between adjacent structural bays, especially where panel-to-panel connections provide little bending stiffness. If panel joints are explicitly included in FEA models, the impact of the continuity of the topping stiffness can be understood by assigning the bending stiffness of the topping to every panel joint and observing the change in mode shapes and modal masses. Section 5.5 discusses these topics in detail.

Breaking continuity of elements can also protect areas that are particularly sensitive to vibration. These areas can be isolated by splitting the floor plate into simply-supported sections. Isolated floors can be helpful in areas where heavy excitation is expected (e.g., gyms/exercise studios) or where sensitive equipment will be used.

3.5.2 Architectural Considerations

The vibration of a floor depends on walking frequency and path. Generally, the fastest walking frequencies occur in open spaces and circulation corridors, and people immediately adjacent to these areas will experience the most impact from associated vibrations. The impacts of walking frequency and stride length are discussed further in Section 3.6.

One approach to mitigating the potential for unsatisfactory vibration levels is to avoid mid-span locations and other locations of peak modal displacement in the positioning of corridors and circulation areas. Another is to break up corridor lengths to reduce walking frequency.

Partitions and nonstructural walls offer another opportunity to improve vibration performance because they can impart additional mass and stiffness to the floor. Considerations for modeling these effects are discussed further in Chapter 5.

3.6 Excitation Parameters

When performing modal response analysis consistent with the recommendations of this guide, quantification of dynamic loading due to footfalls is required. Excitation parameters include walking frequency, walker weight, and stride length, as discussed below.

3.6.1 Walking Frequencies

Walking frequency (f_w) has a significant influence on the dynamic response of a floor. If one of the natural frequencies of the floor system is harmonic with the walking frequency, resonance can occur. Likewise, higher walking frequencies correspond to larger individual footfall forces, increasing the floor's resonant and transient response. For example, Figure 3-6 shows an example of the resonant acceleration response of a floor over a range of walking frequencies and damping values. In this example, the floor has resonance with walking around 2.1 Hz and 2.25 Hz.

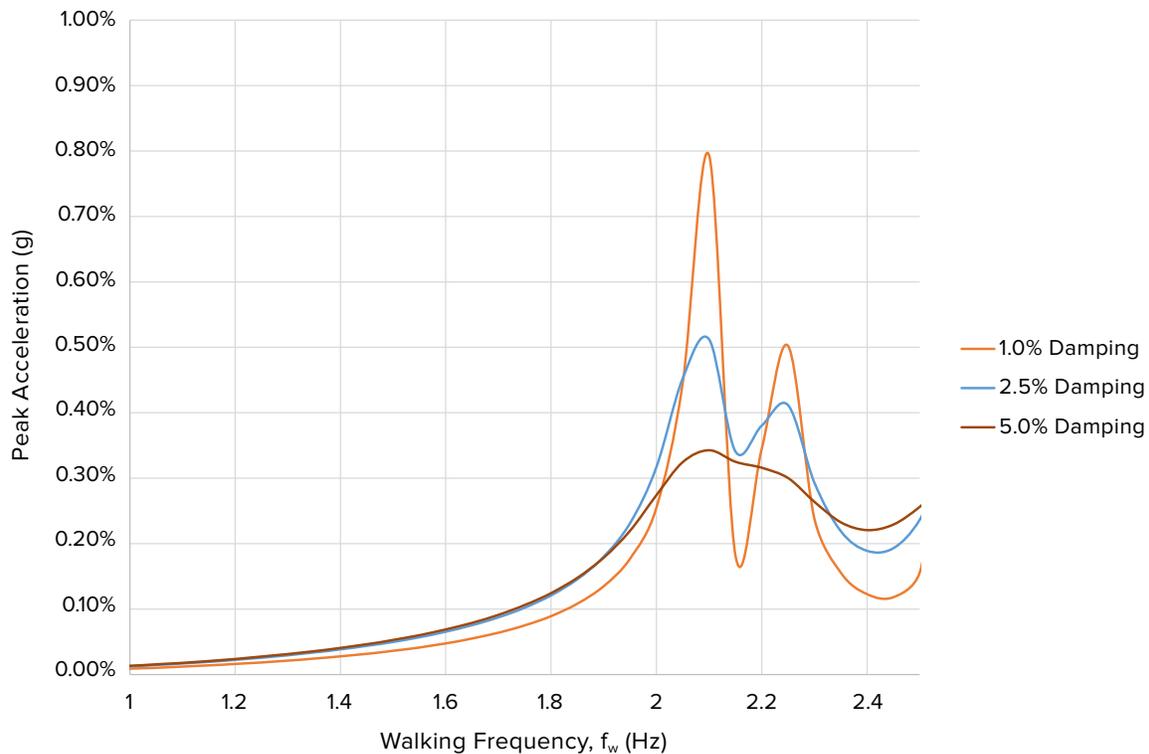


FIGURE 3-6: Example resonant acceleration response to walking frequency and damping

Human walking frequencies generally fall between 1.25 Hz and 2.1 Hz. Higher frequencies will occur in larger spaces and circulation areas which promote faster walking, while lower walking frequencies will occur in enclosed spaces where people walk more slowly. A range of walking frequencies, following AISC Design Guide 11, is outlined in Table 3-7.

TABLE 3-7: Example walking frequencies (AISC Design Guide 11)

Walking Speed	Walking Frequency (Hz)	Steps Per Minute (SPM)	Potential Occupancies
Very slow (uncommon)	1.25	75	Laboratories, surgical theaters
Slow	1.6	95	Bedrooms, hotel rooms
Moderate	1.85	110	Residential living areas, office work areas
Fast	2.1	125	Corridors, shopping malls, airports

PRACTICAL TIP
 If you are unsure whether a walking frequency is appropriate, use a metronome to walk at the target pace by setting beats per minute to the appropriate SPM.

From AISC Design Guide 11: “Very slow walking applies to areas with one or two walkers and limited walking paths; examples are laboratories with fewer than three workers and medical imaging rooms. Slow walking applies to areas with three or four potential walkers and limited walking paths. Moderate walking applies to busy areas with fairly clear walking paths. Fast walking applies to areas with clear walking paths, such as corridors.”

Running, while not often considered in normal occupancy design, can have footfall frequencies as high as 4.0 Hz and may need to be considered in special applications.

CCIP-016 has higher recommended maximum walking frequencies of 2.0 Hz within office bays and residential rooms and 2.5 Hz in corridor and circulations zones in any building.

As a starting point, the authors of this guide recommend selecting a walking speed and occupancy per Table 3-7 and performing an initial evaluation of the floor. A sensitivity analysis can then be performed to assess the degree of precision required in the refinement of this parameter.

3.6.2 Weight of Walker

The dynamic force applied to both low- and high-frequency floors scales proportionally with the static weight of the design walker (P). While there is currently no agreed-upon standard walking weight, this guide uses 168 lbs, which is consistent with the recommendations in AISC Design Guide 11 and marginally greater than the CCIP-016 recommendation of 158 lbs. Engineering judgment may be used to justify a different walker weight if desired.

3.6.3 Stride Length

Stride length (l) impacts the number of strides that can be taken over a given floor length. This parameter can in turn influence whether a low-frequency floor can reach a fully resonant steady-state response. If only a small number of steps is taken, the vibration amplitude will be less than the full resonant response. Values for stride length are typically in the range of 2 to 3 ft.

3.6.4 Resonant Footfall Loading Function

Resonant loading is concerned with the response of the structure after multiple footfall events, so the shape of the footfall loading function significantly affects vibration calculations. The load function form is defined first with a sharp increase in force as the heel makes contact, followed by a period of relaxation as the walker transitions through his or her step and pushes off the floor; this series of events can be represented with a periodic function, as shown for ascending and descending stairs in Figure 3-7. Significant research has been focused on determining accurate forcing function magnitudes and shapes to represent typical walking activity (Kerr, 1998).

To simplify the calculations used in modal analysis techniques, the resonant forcing function of an individual walking across a floor plate is typically represented using a Fourier series decomposition of the forcing function (i.e., a collection of sinusoidal functions that sum together to equal the forcing function). Recommended values of the harmonic coefficients and their intended use are presented in Section 4.3.1.

3.6.5 Transient Footfall Loading Function

For transient vibration, only the force from a single footstep is needed for analysis. In this case, excitation due to walking is most commonly represented with an equivalent impulse to excite the floor, and the resulting dynamic response is calculated directly. Using the same walking data used for low frequency floors, researchers have developed impulsive excitation forces calibrated to both the walking frequency and the structure's natural frequencies (CCIP-016). Recommendations consistent with AISC Design Guide 11 are presented in detail in Section 4.3.2.

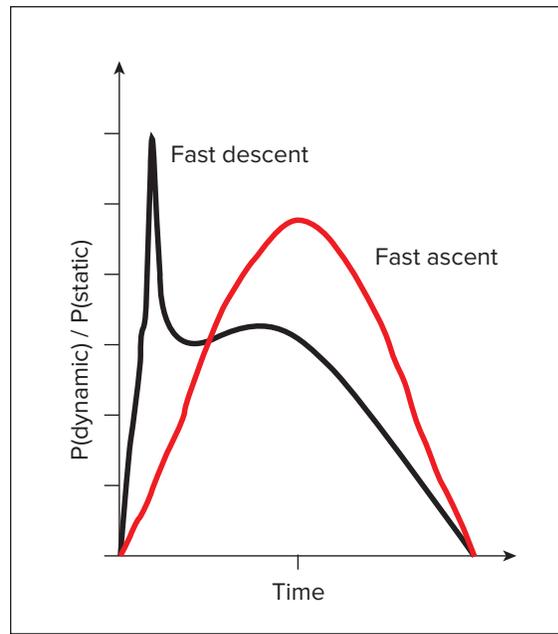


FIGURE 3-7: Dynamic loading function for continuous excitation due to walking (Kerr, 1998)

3.7 Floor Vibration Performance Targets

Human tolerance of vibration is highly subjective and varies with the activity of the observer. However, when dynamic model-based methods are used, specific acceleration and/or velocity values are selected as quantitative performance targets. Guidance for targets representing a low likelihood of occupants complaining of discomfort (“low probability of adverse comment”) varies widely between occupancies and user activities. A vibration level that is not worthy of comment when walking through a shopping mall might be completely unacceptable if felt when lying down to rest. While the perception thresholds for vertical accelerations are well documented (see Section 2.4), what is acceptable in a building will vary from person to person and potentially between regions and countries.

3.7.1 Review of Established Performance Targets

Performance targets from AISC Design Guide 11, CCIP-016 and ISO 10137: Bases for design of structures – Serviceability of buildings and walkways against vibrations (ISO, 2007), are provided in the following sub-sections. Separate targets are typically provided for low and high-frequency floors, with the target provided as acceleration for low-frequency floors (i.e., $f_n < 8$ to 10 Hz) and velocity for high-frequency floors (i.e., $f_n > 8$ to 10 Hz).

3.7.1.1 Resonant Response Analysis (Low-Frequency Floors)

The ISO 10137 standard suggests the performance targets in Table 3-8 for accelerations experienced as continuous or intermittent vibrations.

CCIP-016 and similar vibration guides use a response factor, R, to normalize the acceleration values by the limit of perception, so the magnitude of acceleration is often reported on a multiplier of the baseline ISO curve. In other words, the Response Factor R is the measured or predicted response divided by the threshold of perception. Based on practical experience, CCIP-016 provides a set of suggested performance targets for acceleration response to excitations from a single walker, which are summarized in Table 3-9. For residential and office occupancies, the CCIP-016 acceleration targets for walking excitations are higher than those recommended in ISO 10137.

TABLE 3-8: Floor performance targets from ISO 10137

Place	RMS Acceleration Target in 4-8 Hz Range	Target Response Factor, R
Workshops	0.40% g	8
General office (schools, office)	0.20% g	4
Residential (day)	0.10%- 0.20% g	2- 4
Quiet office (open plan)	0.10% g	2
Residential (night)	0.07% g	1.4
Critical work areas	0.05% g	1

The method for low-frequency floors in CCIP-016 predicts the peak acceleration, not RMS acceleration. The baseline limit of perception for RMS acceleration from the ISO curve is $0.005 \text{ m/s}^2 = 0.05\% \text{ g}$ which corresponds to a sinusoidal peak acceleration of $0.071\% \text{ g}$. The peak amplitude of a sinusoidal motion is $\sqrt{2}$ times the RMS amplitude.

TABLE 3-9: Floor acceleration performance targets from CCIP-016

Place	Equivalent Peak Acceleration	RMS Acceleration Target in 4-8 Hz Range	Target Response Factor, R
Commercial buildings including offices, retail, restaurants, airports	0.57% g	0.40%	8
Residential (day)	0.28%- 0.57% g	0.20%- 0.40% g	4- 8
Premium quality office, open office with busy corridors near mid-span, heavily trafficked public areas with seating	0.28% g	0.20% g	4
Residential (night)*	0.20% g	0.14% g	2.8
Hospitals and critical work areas	0.071% g	0.05% g	1

* CCIP-016 suggests that this more restrictive target may not be necessary.

For acceleration-based performance targets, AISC Design Guide 11 provides a set of peak acceleration performance targets as shown in Table 3-10.

TABLE 3-10: Floor acceleration performance targets from AISC Design Guide 11

Place	Peak Acceleration Target in 4-8 Hz Range	Equivalent RMS Acceleration	Equivalent Response Factor, R
Outdoor pedestrian bridges	5% g	3.5% g	70
Residential (day)	1.5% g	1.05% g	21
Offices, residences, quiet areas	0.5% g	0.35% g	7

While AISC Design Guide 11 does not use the response factor, R, the equivalent response factor consistent with CCIP-016 is presented to aid in comparing the different targets.

Comparing the various targets discussed above, the imprecise nature of selecting performance criteria is apparent. For example, between the three sources detailed, the peak acceleration target for offices varies from 0.14% g for a quiet office in ISO 10137 to 0.50% g for general offices in AISC Design Guide 11. The range of peak acceleration targets for residential occupancies is even larger from 0.10% g in ISO 10137 (at night) to 0.57% g in CCIP-016.

3.7.1.2 Transient Response Analysis (High-Frequency Floors)

For high-frequency floor performance, AISC Design Guide 11 Chapter 6 and CCIP-016 both use an RMS velocity measure of floor vibration. The threshold of perception on RMS velocity in CCIP-016 is stated as 1×10^{-4} m/s and is used as the normalizing factor for the response factor, R, for velocity performance. The velocity performance targets in CCIP-016 and AISC Design Guide 11 for occupant comfort are shown in Table 3-11 and Table 3-12, respectively.

TABLE 3-11: Floor velocity performance targets from CCIP-016

Place	RMS Velocity Target*	RMS Velocity Target*	Target Response Factor, R
Commercial buildings including offices, retail, restaurants, airports	8×10^{-4} m/s	32,000 mips	8
Residential	$4-8 \times 10^{-4}$ m/s	16,000-32,000 mips	4-8
Premium quality office, open office with busy corridors near mid-span, heavily trafficked public areas with seating	4×10^{-4} m/s	16,000 mips	4

*RMS velocity performance targets where f_n is 8 Hz or greater

TABLE 3-12: Floor velocity performance targets from AISC Design Guide 11

Place	RMS Velocity Target*	RMS Velocity Target*	Equivalent Response Factor, R
Ordinary workshops	8×10^{-4} m/s	32,000 mips	8
Offices	4×10^{-4} m/s	16,000 mips	4
Residences	2×10^{-4} m/s	8,000 mips	2
Hospital patient rooms	1.5×10^{-4} m/s	6,000 mips	1.5

*RMS velocity performance targets where f_n is 8 Hz or greater

The velocity targets from the two guides above illustrate how human tolerance to vibration depends on the occupancy; spaces that are expected to be busier and with higher levels of activity tend to have greater target velocities than spaces that are expected to be quiet and sparsely populated. In comparing the guides, both specify 16,000 micro-in./s for offices; however, they differ by a factor of between 2 and 4 for residences.

3.7.2 Selection of Performance Targets

Because of the prevalence of AISC Design Guide 11 in U.S. office design, this guide recommends using the vibration performance targets in that standard. For open-plan office occupancies, it is common to use a peak acceleration target of 0.5% g for low-frequency floors and an RMS velocity target of 16,000 micro-in./s for high-frequency floors. The authors acknowledge that these two metrics do not correspond to the same R factor in the CCIP-016 methods. For a premium office, a more stringent RMS velocity target such as 8,000 micro-in./s may be appropriate, keeping in mind the influence of the range of walking frequencies on the predicted performance.

For residential occupancies, AISC Design Guide 11 recommends target performance values of 0.5% g peak acceleration and 8,000 micro-in./s RMS velocity. These targets may be appropriate for high-performing open framing systems; however, mass timber floor systems for residential occupancies often use bearing wall framing, not post-and-beam framing. In such conditions, CCIP-016 suggests a performance target equivalent to 16,000 to 32,000 micro-in./s, noting that the upper end of the range would not be appropriate for condominiums or luxury apartments.

The predicted acceleration or velocity response of a floor provides an idea of performance, but small variances are generally not noticeable and don't represent the difference between a well- or poor-performing floor; small variations (10 to 20%) are not perceptible to humans. Some standards note that, if the magnitude of the vibration response doubles the recommended limit, adverse comment may result, and quadrupling the response factor limit may significantly increase the level of adverse comment (BS 6472). In many cases, to reduce a floor's response by half, significant design changes with cost implications and/or layout changes may be required.

This highlights the difficulty of assigning specific acceptance criteria for vibration. Selection of vibration performance targets, at least qualitatively, should be discussed with the client early in schematic design to understand and manage expectations. The suggested performance targets are summarized in Table 3-13.

TABLE 3-13: Suggested performance targets

Place	Peak Acceleration Target	RMS Velocity Target
Offices or residences	0.5% g	16,000 - 32,000 mips
Premium offices or luxury residences	0.3% g	8,000 - 16,000 mips

As an example of performance targets selected for a mass timber system design, in the Timber Tower Research Project by Skidmore, Owings & Merrill (2013), the TCC floor system was designed to achieve a target RMS velocity response of 16,000 micro-in./s in the open living areas for a moderate walking frequency (1.85 Hz), and 8,000 micro-in./s in the sleeping areas for a slow walking frequency (1.6 Hz).

3.7.3 Vibration-Sensitive Facilities

Equipment such as MRI/CT scanners, electron microscopes and operating theaters often have more stringent limits than floors designed for human sensitivities, including at high frequencies. Limits are generally less flexible and often prescribed by the manufacturer in terms of peak velocity or acceleration, narrowband spectral velocity or acceleration, or one-third octave spectral velocity or acceleration to ensure proper functioning and precision. If the specific equipment requirements are unknown, it is common practice to rely on generic velocity criteria (VC) targets. Velocity criteria targets for sensitive equipment, based on AISC Design Guide 11 and CCIP-016, are shown in Table 3-14. To achieve the performance targets for human comfort in patient rooms and surgery facilities, VC-A and VC-B, a frequency-weighted sensitivity is used. (See Figure 2-5.) For VC-C, VC-D and VC-E, a constant sensitivity from 1 to 80 Hz is used. These generic velocity criteria for sensitive equipment are commonly evaluated in one-third octave frequency bands, as discussed in Section 4.3.2. Chapter 9 provides a detailed example calculation of vibration performance verification for a floor that supports vibration-sensitive equipment.

TABLE 3-14: Generic velocity criteria for sensitive equipment

Designation	RMS Velocity Limit	Application
N/A	6,000 mips	Hospital patient rooms
N/A	4,000 mips	Surgery facilities, laboratory robots, bench microscopes up to 100x, operating rooms
VC-A	2,000 mips	Microbalances, optical comparators, mass spectrometers, industrial metrology laboratories, spectrophotometers, bench microscopes up to 400x
VC-B	1,000 mips	Microsurgery, microtomes and cyrotomes for 5-10 μm slices, tissue and cell cultures, optical equipment on isolation tables, bench microscopes greater than 400x, atomic force microscopes
VC-C	500 mips	High-precision balances, spectrophotometers, magnetic resonance imagers, microtomes and cyrotomes for < 5 μm slides, chemotaxs, electron microscopes at up to 30,000x
VC-D	250 mips	Cell implant equipment, micromanipulation confocal microscopes, high-resolutions mass spectrometers, electron microscopes at greater than 30,000x
VC-E	125 mips	Unisolated optical research systems, extraordinarily sensitive systems

4

Vibration Design Methods

This chapter provides guidance on the selection and implementation of appropriate method(s) for design based on specific design scenarios. Methods covered include the simplified formulation from the CLT Handbook, as well as the detailed methods required to implement modal response analysis for resonant and transient-governed floors based on approaches presented in AISC Design Guide 11 and CCIP-016.

4.1 Choosing the Right Methodology

Choosing the right methodology is important, as not all methods are appropriate for use in all configurations. A brief outline of the general approaches is provided in Section 2.5. The various analysis approaches have differing attributes and limitations, which are summarized in Table 4-1.

TABLE 4-1: Selected methods, attributes and limitations

Analysis Method	Design Guide	Attributes	Limitations
Empirical formula	CLT Handbook	Relationship from tested simply-supported, single-span bare CLT	Limited applicability due to extent of tested systems; no consideration of support flexibility; no consideration of different target performance levels
Simplified modal formulas	AISC Design Guide 11, SCI P354, CCIP-016	For wall- or beam-supported systems	Existing methods calibrated to steel or concrete systems and not necessarily appropriate for mass timber floors
Modal response analysis	AISC Design Guide 11, SCI P354, CCIP-016	Accommodates different damping ratios, floor mass, span or fixity conditions, and excitation/response locations	Sensitive to excitation and modeling assumptions
Time history analysis	AISC Design Guide 11, SCI P354, CCIP-016	Directly accounts for time effects (number of strides taken, etc.) and spatial effects (walking path, etc.)	Difficult to implement; high sensitivity to excitation and modeling assumptions; user judgement required to properly select walking paths and other inputs

When considering vibration design of a mass timber floor system, Figure 4-1 can help determine the applicability of the methods described in this guide.

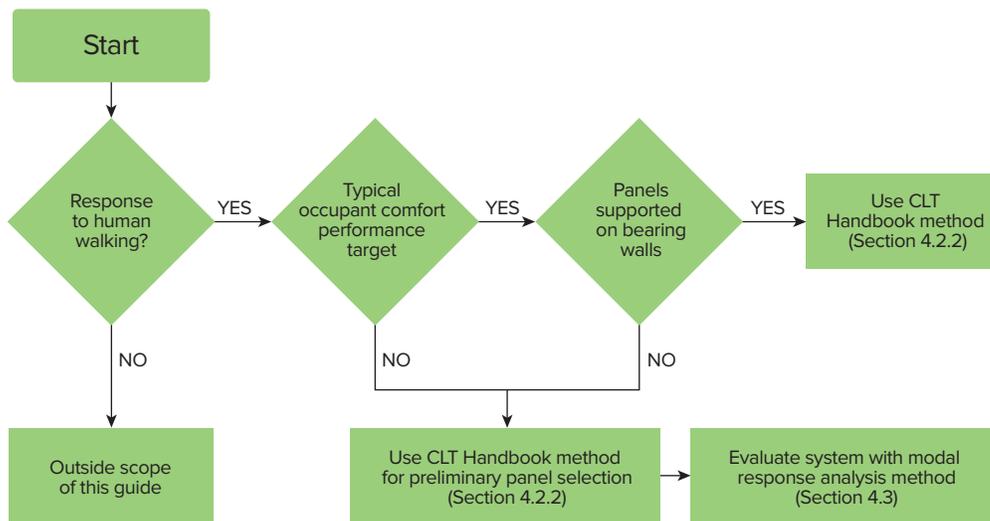


FIGURE 4-1: Vibration design flow chart

4.2 Simplified Procedures

Simplified formulae can be used to calculate a floor’s performance based on underlying simplifications and assumptions. However, they tend to be conservative for floors that are similar to those used in the original development of the formulae. Additionally, these methods should be used with caution when applied outside their original bounds. Although sometimes suitable as a final design, these simplified approaches are typically used for preliminary design to test for major vibration problems in advance of a more detailed analysis.

4.2.1 Estimates of Natural Frequency

Determining the fundamental frequency of the floor system is a helpful starting point for both simplified and more complex analyses, providing insight into the potential susceptibility of the floor to vibrations. The fundamental natural frequency of a simply-supported prismatic beam with a uniform lineal mass, \bar{m} , or uniform lineal weight, w , can be calculated using values with self-consistent units, from:

$$f_n = \frac{\pi}{2L^2} \sqrt{\frac{EI}{\bar{m}}} = \frac{\pi}{2L^2} \sqrt{\frac{gEI}{w}}$$

A convenient reorganization of the above is to approximate the frequency of a single-span system with the following equation:

$$f_n = 0.18 \sqrt{\frac{g}{\Delta}} = \frac{3.54}{\sqrt{\Delta}}$$

Where:

g is the acceleration of gravity, 386.1 in./s²

Δ is the deflection to the expected weight, in.

The expected weight is the floor self-weight, plus the superimposed dead load, plus the portion of the live load expected to be present, as discussed in Section 3.1.

As discussed in Section 2.5.2, AISC Design Guide 11 and the NBCC recommend against floors with $f_n < 3$ Hz and 4 Hz, respectively. Conversely, floors are unlikely to have vibration problems if $f_n > 15$ Hz (AISC, 2016). Floors with f_n within these limits (i.e., 3-4 Hz $< f_n < 15$ Hz) warrant more detailed analysis.

An approach to estimating a lower bound of the fundamental frequency of a floor system of panels supported on beams can be made using Dunkerley's method accounting for the natural frequencies of the hierarchy of supporting elements. (Blevins, 2016)

$$\frac{1}{f_n^2} = \frac{1}{f_{\text{panel}}^2} + \frac{1}{f_{\text{beam}}^2} \left[+ \frac{1}{f_{\text{girder}}^2} \dots \right]$$

The natural frequency of systems incorporating multiple flexible components will be less than the lowest natural frequency of any component. This fact highlights the risks of using assumptions such as minimum natural frequency of panels without considering the effect of the flexibility of the supporting beams.

4.2.2 CLT Handbook Method

The U.S. CLT Handbook and Canadian CLT Handbook, 2nd Edition present a recommended span limit for acceptable performance of CLT panels to typical walking excitations. This method was developed through laboratory testing of single-span, simply-supported, bare CLT panels supported on walls with the CLT spanning in the major strength direction of the panel. Based on subjective observations from multiple people walking and sitting, an empirical formula provides a recommended span limit, L_{lim} , based on the bending stiffness, EI_{eff} , in units of lbf-in²/ft, specific gravity, $\bar{\rho}$, and cross-sectional area, A , of a 1-ft-wide strip of panel in units of lbf-in²/ft.

The recommended span limit from the 2019 Edition of the Canadian CLT Handbook, converted to U.S. customary units is:

$$L_{\text{lim}} \leq \frac{1}{12.05} \frac{(EI_{\text{eff}})^{0.293}}{(\bar{\rho}A)^{0.122}} \text{ (ft)}$$

The published value of $EI_{\text{eff},f,0}$ for the CLT should be used for EI_{eff} . Prior versions of this formula used the apparent stiffness, EI_{app} , instead of the effective stiffness, EI_{eff} .

The specific gravity, $\bar{\rho}$, used in this equation is an estimate of the weight of the CLT panel. This may differ from the specified specific gravity, G , for the lumber used in the construction of the CLT. The specified specific gravity, G , is used for connection design and based on oven-dry wood properties. For the CLT Handbook method, it is appropriate to use the specific gravity based on the actual panel weight or density provided by the manufacturer. Given the density, ρ , the specific gravity is calculated by dividing by the weight of water, 62.4 lb/ft³. For example, with a CLT density of 32.0 lb/ft³, the specific gravity is calculated as $\bar{\rho} = 32.0 / 62.4 = 0.51$. Where the weight of the panel, w , is available in lb/ft², an alternative form of the recommended span limit can be written as:

$$L_{\text{lim}} \leq \frac{1}{13.34} \frac{(EI_{\text{eff}})^{0.293}}{(w)^{0.122}} \text{ (ft)}$$

Where the panel density or weight is not available from the manufacturer, the specific gravity can be estimated using a moisture-content-to-density relationship found in the NDS Supplement Section 3.1.3, using the moisture content (MC) at time of manufacturing (approximately 12%), and the specified specific gravity of the species group of wood found in the NDS Supplement Table 4.

As this method depends only on bare panel properties, the recommended span limit can be calculated for a panel without consideration of project specifics. This approach does not depend on variations in damping, the weight of additional floor components or contents, multi-span conditions, or the selection of a specific target performance level.

The subjective tests on which the CLT Handbook method is based used 5-ply and 7-ply CLT layups, and the equation does not account for changes in the relative stiffness between the major spanning direction and minor spanning direction of the panels. While the relative stiffness between the major and minor directions can have a noticeable impact on the performance of a floor system, this impact is not accounted for in the CLT Handbook method. Therefore, this method may have a minor bias to be relatively conservative for 9-ply CLT layups of alternating layers. Conversely, the CLT Handbook method may have a minor bias to be relatively unconservative for typical 3-ply CLT layups. Similarly, the method may have a bias to be relatively unconservative for floor panels with little inherent minor axis stiffness, such as those made with NLT, DLT and GLT.

A sample calculation of the CLT Handbook method is performed below for a 4 1/8-in.-thick panel layout of CLT grade V2. The properties t_p and EI_{eff} are found in PRG 320-19 and the specified specific gravity, G , is found in NDS Supplement Table 4A for visually-graded spruce-pine-fir (SPF).

TABLE 4-2: CLT Handbook example panel properties

CLT Layout	t_p (in.)	EI_{eff} ($\times 10^6$ lbf-in ² /ft)	G
V2	4 1/8	95	0.42

Estimating the in-service specific gravity at 12% moisture content using NDS Supplement Section 3.1.3:

$$\bar{\rho} = \left[\frac{G}{1 + G(0.009)(MC)} \right] \left[1 + \frac{MC}{100} \right] = \left[\frac{0.42}{1 + 0.42(0.009)(12)} \right] \left[1 + \frac{12}{100} \right] = 0.45$$

$$A = t_p * 12 \text{ in.} = 4 \frac{1}{8} \text{ in.} * 12 \text{ in.} = 49.5 \text{ in.}^2$$

$$L_{lim} \leq \frac{1}{12.05} \frac{(EI_{eff})^{0.293}}{(\bar{\rho} A)^{0.122}} = \frac{1}{12.05} \frac{(95,000,000)^{0.293}}{(0.45 * 49.5)^{0.122}} = 12.4 \text{ ft}$$

Alternatively, using the area weight of the CLT panel:

$$w = \bar{\rho} \rho_{H2O} t_p = 0.45(62.4 \text{ pcf})(4.125 \text{ in.}) \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right) = 9.65 \text{ psf}$$

$$L_{lim} \leq \frac{1}{13.34} \frac{(EI_{eff})^{0.293}}{(w)^{0.122}} = \frac{1}{13.34} \frac{(95,000,000)^{0.293}}{(9.65)^{0.122}} = 12.4 \text{ ft}$$

The vibration-controlled spans of all PRG 320-19 basic grades and layouts, based on assumed in-service moisture content of 12% to estimate panel weight, are presented in Table 4-4 at the end of this chapter.

Developers of the CLT Handbook method have provided the following recommendations for specific cases outside the basic assumptions:

- This method may be directly used for CLT panels when a concrete layer is applied on the CLT where the weight of the concrete layer is not greater than twice the weight of the CLT.
- This method may be applied to individual spans of multi-span CLT panels.
- For multi-span panels “with a nonstructural element that is considered to provide enhanced vibration effect,” the recommended span limit can be increased up to 20%, provided the span is less than 26.2 ft (8 meters)

For the 20% increase, construction with applied gypsum ceilings, nonstructural interior partition walls or other features that provide damping or floor-to-floor restraint can be considered to provide enhanced vibration effects. The Canadian CLT Handbook states the 20% increase doesn’t apply to floors with concrete topping; however, additional testing and field experience has led to the relaxation of this limitation. Therefore, this guide recommends that the increase in the recommended span limit up to 20% for multi-span panels can be applied even with a concrete topping.

The U.S. CLT Handbook provides a reduction in the basic span limit when topping layers weighing greater than 20 psf are applied. The Canadian CLT Handbook recommends a 10% reduction in the calculated span limit when the weight of the topping layers is greater than twice the weight of the CLT. This guide recommends following the Canadian CLT Handbook limit.

In practice, the CLT Handbook method has been shown to result in well-performing floors in applications where the panels are supported on bearing walls. Experience using this method, mostly for residential applications, is documented in the Canadian CLT Handbook. The authors are also not aware of any poor-performing floors meeting these conditions. Furthermore, individual engineers and manufacturers are known to go beyond the recommended span limits based on their experience with installed CLT floor systems.

The CLT Handbook method does not capture the reduced vibration performance due to flexibility of a supporting frame. For floors supported on beams, the Canadian CLT Handbook proposes an empirical equation that relates a supporting beam span to a target stiffness for the CLT Handbook method to remain valid; however, this relationship does not account for factors such as the tributary area or loading to the beams or the cumulative flexibility of panel supported by purlins supported by girders, so judgment is required when applying its result.

Even in conditions with flexible supports, the CLT Handbook method is a good starting point for selecting candidate floor panels. The modal response analysis methods described in the following sections can be followed to predict the impact of supporting framing on the total floor performance in greater detail.

4.3 Modal Response Analysis Methods

Modal response analysis methods combine the responses in multiple individual modes of vibration determined from FEA into a single-floor response through a process of modal superposition. Modal response analysis is more flexible than the simplified calculation methods presented in Section 4.2 and can be performed for both resonant and transient analyses. This flexibility requires the designer to account for many considerations including damping, boundary conditions, anticipated range of walking frequency and possible composite action. The accuracy of the results depends significantly on the quality of input data. The target performance criteria also vary as discussed in Section 3.7.

Performing a modal analysis requires an accurate FEA model to predict the dynamic response of the structure. Chapter 5 provides information on modeling approaches. While many FEA programs can be used to perform the dynamic modal analysis, few perform the data processing needed to combine the modal results into the estimate of floor vibration performance. If the FEA program does not calculate the combined response according to CCIP-016, AISC Design Guide 11 and/or SCI P354 methods, engineers use independent post-processing of the data to accomplish this.

One common and simplifying analysis assumption is co-location of the source of excitation and the observation of excitation. That is, the walker and observer are assumed to be at the same point. However, if architectural layouts and anticipated walking paths are well understood, the response at points of interest (e.g., in a residential suite or corner office) can be assessed for excitations occurring at different locations (e.g., in an adjacent corridor).

It is technically possible to perform the modal superposition process using hand calculations; however, this is not practical. Unless vibration-specific modal superposition methods are implemented in the structural analysis package, a post-processor is created to perform the large number of calculations with the modal properties from the FEA as input values. Refer to Chapter 6 for a detailed description of post-processing.

4.3.1 Resonant Response Analysis (Low-Frequency Floors)

Resonant response analysis is applicable for low-frequency floors where resonance due to walking excitations may cause the floor's vibration response to be noticeable or even excessive. This can occur when the fundamental frequency of the floor, f_n , is close to any harmonic of the walking frequency, up to the fourth harmonic. CCIP-016 recommends:

If $f_n < 4f_w + 2$ Hz, perform resonant response analysis

This upper frequency limit recommendation varies with walking frequency; however, an approximate threshold of 8 to 9 Hz is sometimes used (e.g., AISC Design Guide 11).

For resonant response analysis, CCIP-016 recommends considering all modes with modal frequencies, f_m , up to 15 Hz. A resonant response analysis performed over a grid of points over a whole floor system can result in a plot of predicted acceleration values as shown in Figure 4-2. Such plots are referred to as heat-maps of the response.

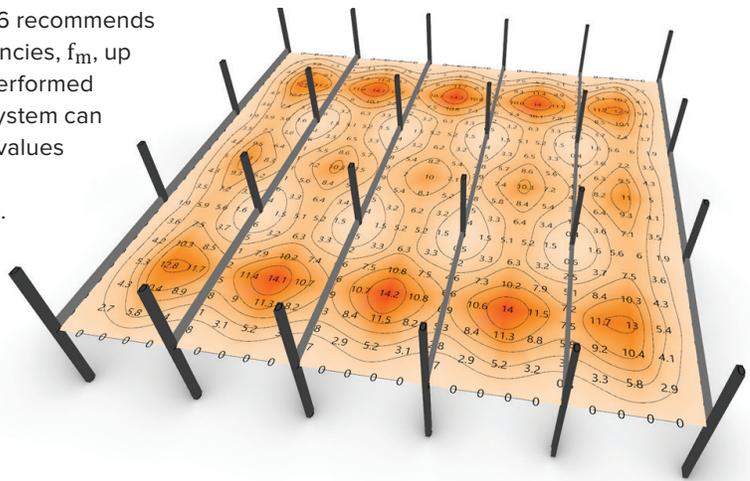


FIGURE 4-2: Resonant modal response analysis results across a typical floorplate (CCIP-016 R factors shown)

4.3.1.1 Excitation

The range of considered walking frequencies is chosen based on the expected occupancy of the floor as discussed in Section 3.6. Often a harmonic of one walking frequency considered matches a modal frequency and dominates the response (e.g., the third harmonic of 2.0 Hz walking frequency creates resonance on a 6.0 Hz floor). To capture the potential resonance response values, the full range of walking frequencies is used to calculate the response vs. frequency. Figure 4-3 shows an example of the results for a floor with resonant response calculated for a number of walking frequency increments. Because the calculated response changes rapidly near peaks, it is recommended that a relatively small frequency increment is used, such as 0.025 Hz or smaller, to adequately capture the peaks in response. Figure 4-3 demonstrates how using too large an increment in the walking frequency for analysis can lead to missing the peaks of the resonance.

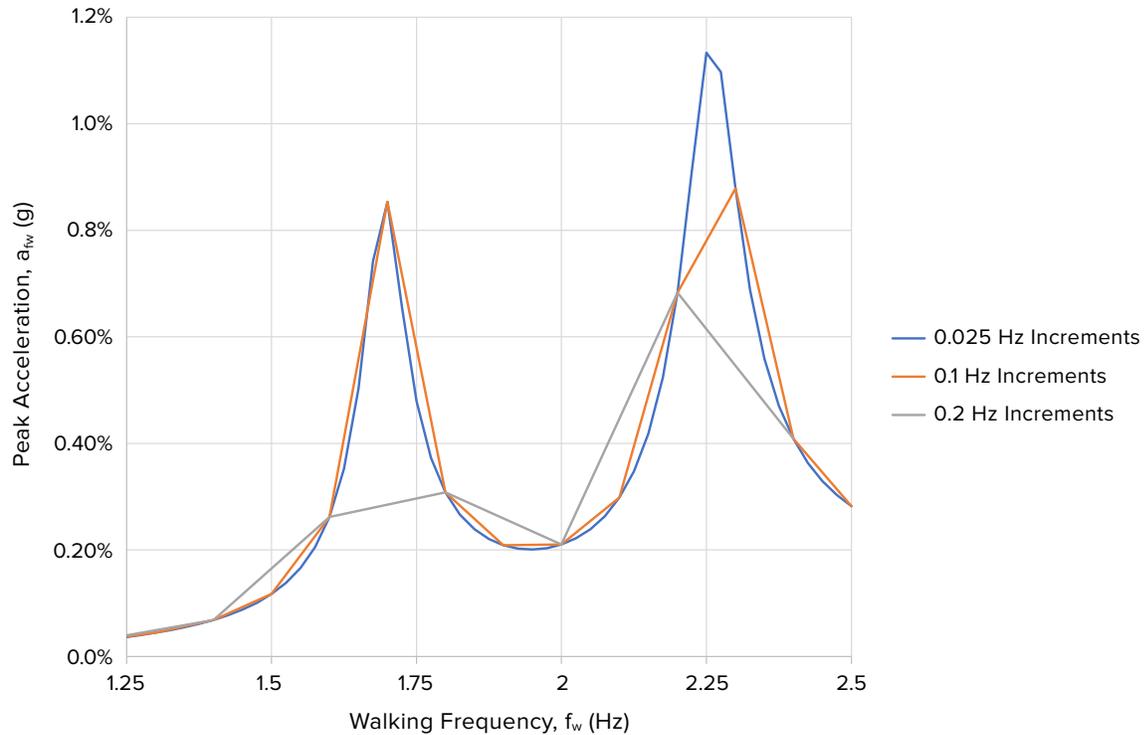


FIGURE 4-3: Acceleration vs. walking frequency calculated with different frequency increments

The analysis for a specific walking frequency, f_w , begins by calculating, for each mode, a harmonic force, F_h , for the first four harmonics of the walking frequency. This force represents the amplitude of the forcing function derived empirically from tests of walkers on instrumented platforms. F_h is the dynamic component of the total force applied by the walker to the floor (i.e., it is the total load minus the static weight of the walker). F_h is expressed as a harmonic coefficient, α_h , multiplied by the static weight (P) of the walker.

$$F_h = \alpha_h P$$

Design-level harmonic coefficients, α_h , from CCIP-016 are presented in Table 4-3. The harmonic coefficients, also called dynamic load factors (DLF), from CCIP-016 are based on a larger and later data set than that used to derive the harmonic coefficients in AISC Design Guide 11. The DLF is the dynamic force that is exciting a given harmonic expressed as a fraction of the static weight of the walker as a function of the harmonic frequency, $f_h = h f_w$. The DLF tends to be greatest for the first harmonic and to decrease for higher harmonics.

The relationships between the harmonic force and the harmonics of the walking frequency using a walker weight, P, of 168 lbf are shown visually in Figure 4-4.

TABLE 4-3: Design values of footfall harmonic coefficients (CCIP-016 Table 4.3)

Harmonic number, h	Harmonic Coefficient, α_h
1	$0.41(f_h - 0.95) \leq 0.56$
2	$0.069 + 0.0056f_h$
3	$0.033 + 0.0064f_h$
4	$0.013 + 0.0065f_h$
$h > 4$	0

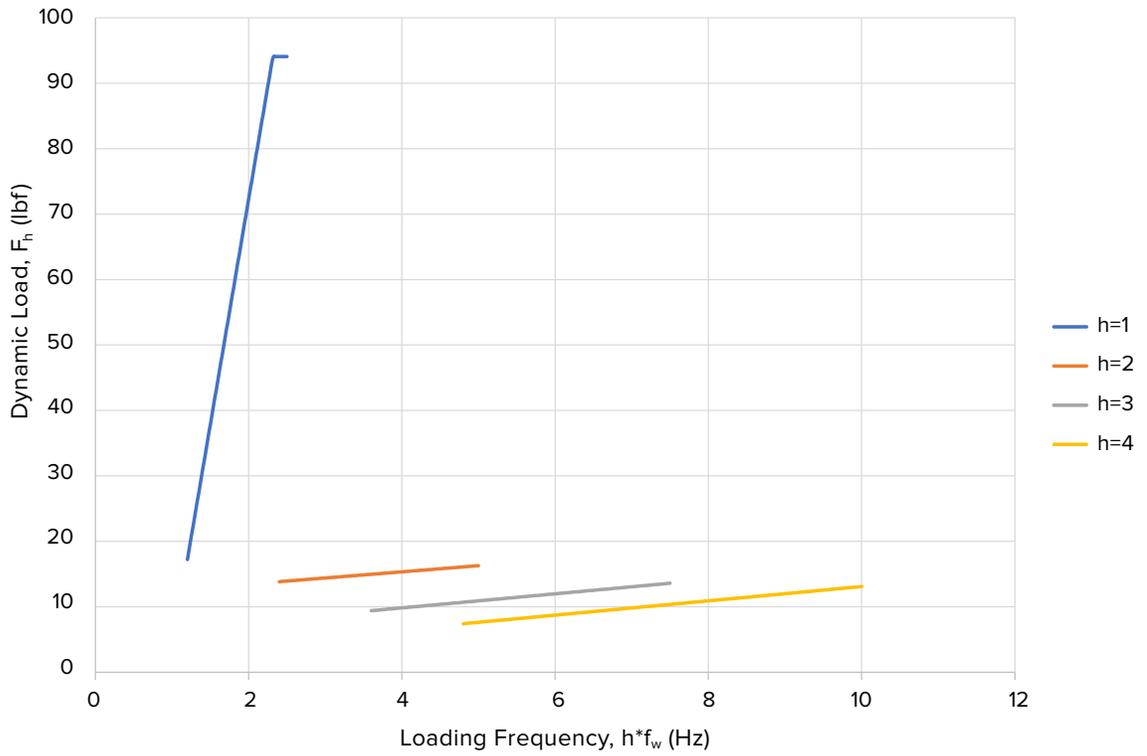


FIGURE 4-4: Dynamic load vs. frequency

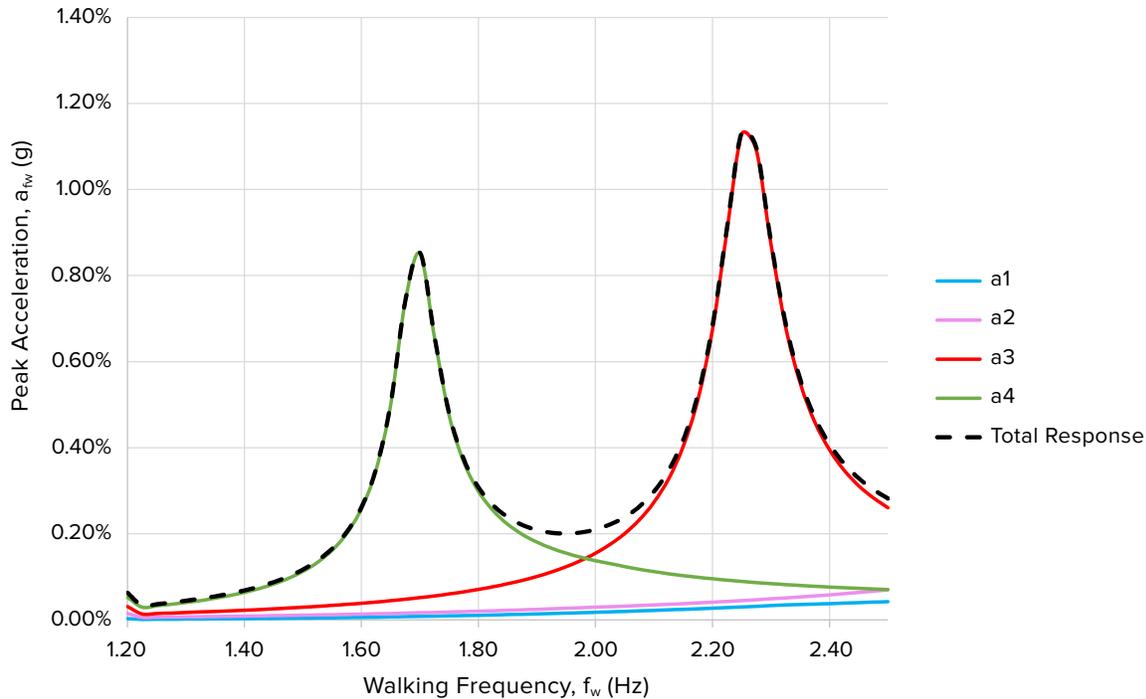


FIGURE 4-5: Harmonic components of total acceleration response

The harmonic loading functions estimate the full steady-state resonant response. Depending on the walking path and space planning, it is possible that walkers have crossed and exited the space before the appropriate number of steps (i.e., loading cycles) have taken place to achieve steady-state response. CCIP-016 proposes a sub-resonant correction factor, $\rho_{h,m}$, to account for this effect; however, it is not usually found to be influential in floors and can be conservatively taken as unity.

$$\rho_{h,m} = 1 - e^{-2\pi\zeta_m N_h}$$

Where:

ζ_m is the modal damping ratio

N_h is the calculated number of loading cycles, e.g., steps, from:

$$N_h = 0.55h \frac{L}{l}$$

L is the span length

l is the stride length

4.3.1.2 **Modal Response Combination**

Combining the above, the resonant acceleration at the point of interest for each mode and harmonic are calculated.

The steady-state acceleration response is at the same frequency as the forcing function, but its magnitude and phase shift compared with the forcing function are different for each mode of the structure. For mathematical convenience, the response is expressed in terms of real and imaginary components of acceleration, instead of magnitude and phase. This is a common signal processing technique that uses the complex plane as a way of easily visualizing and combining the magnitude and phase information of multiple responses.

For a given mode and harmonic of the walking frequency, the resonant response accelerations are

$$a_{\text{real,h,m}} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{\hat{m}_m} \frac{A_m}{(A_m^2 + B_m^2)}$$

$$a_{\text{imag,h,m}} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{\hat{m}_m} \frac{B_m}{(A_m^2 + B_m^2)}$$

Where:

$$A_m = 1 - \left(\frac{f_h}{f_m}\right)^2$$

$$B_m = 2\zeta_m \frac{f_h}{f_m}$$

and:

h is the harmonic number (1 to 4)

m is the mode number

f_m is the natural frequency of the mode under consideration (up to 15 Hz)

ζ_m is the modal damping ratio

\hat{m}_m is the modal mass

$\mu_{e,m}$ is the modal amplitude (calculated consistently with \hat{m}_m) at the excitation location

$\mu_{r,m}$ is the modal amplitude (calculated consistently with \hat{m}_m) at the receiver location
(same as $\mu_{e,m}$ when considering co-located)

$\rho_{h,m}$ is the correction factor for sub-resonant response

For a single harmonic, the real and imaginary components of the modal responses are summed separately across all modes (N in total) considered.

$$a_{\text{real,h}} = \sum_{m=1}^N a_{\text{real,h,m}}$$

$$a_{\text{imag,h}} = \sum_{m=1}^N a_{\text{imag,h,m}}$$

The real and imaginary accelerations are then combined to determine the acceleration response for the walking frequency harmonic in question.

$$a_h = \sqrt{a_{\text{real,h}}^2 + a_{\text{imag,h}}^2}$$

For the given walking frequency, this acceleration of resonance is calculated for each of the first four harmonics.

4.3.1.3 *Evaluating Response*

Following the CCIP-016 guidelines, to calculate the predicted total peak acceleration response, a_p , for a walking frequency, the acceleration response from the four harmonics can be combined using a square root sum of the squares method.

$$a_p = \sqrt{\sum_{h=1}^4 a_h^2} = \sqrt{a_1^2 + a_2^2 + a_3^2 + a_4^2}$$

However, as described in Section 2.4, human perception of acceleration is not constant at all frequencies. Thus, when designing for human comfort, a frequency-weighted peak acceleration, $a_{p,fw}$, is calculated to compare with the selected constant performance target. Each harmonic response is weighted based on the harmonic frequency:

$$a_{h,fw} = a_h \times \begin{cases} \frac{\sqrt{f_h}}{2} & \text{if } f_h < 4 \text{ Hz} \\ 1 & \text{if } 4 \text{ Hz} \leq f_h \leq 8 \text{ Hz} \\ \frac{8}{f_h} & \text{if } f_h > 8 \text{ Hz} \end{cases}$$

Then the harmonic responses combined:

$$a_{p,fw} = \sqrt{\sum_{h=1}^4 a_{h,fw}^2} = \sqrt{a_{1,fw}^2 + a_{2,fw}^2 + a_{3,fw}^2 + a_{4,fw}^2}$$

These calculations are then repeated for each walking frequency evaluated in the range of interest and the maximum value of a_p or $a_{p,fw}$ over the range of walking frequencies is compared to the performance targets selected in Section 3.7.

4.3.2 Transient Response Analysis (High-Frequency Floors)

Performance criteria for high-frequency floors is generally based on velocity, rather than acceleration. Floor systems with a natural frequency greater than four times the highest considered walking frequency (i.e., $f_n \geq 4f_w$) can be governed by transient rather than resonant response. Where the fundamental frequency falls within 2 Hz of the fourth harmonic of loading (i.e., $4f_w < f_n < 4f_w + 2 \text{ Hz}$), the floor is transitional between resonant and transient governed, and it is recommended to perform both resonant and transient response analysis.

4.3.2.1 Excitation

In transient response calculations, the walking load is converted to an effective impulse dependent on the floor system's walking frequency and modal frequency. This equation will produce the maximum impulse at the fastest walking frequency, and the performance evaluation of high-frequency floors needs to consider only the maximum walking frequency.

$$I_{\text{eff},m} = \frac{P}{17.8} \frac{(f_w)^{1.43}}{(f_m)^{1.3}} \text{ [lbf-s]}$$

4.3.2.2 Modal Response Combination

Based on the impulse function, a peak velocity is calculated for each mode with a frequency up to two times the fundamental frequency, f_n . For example, if the fundamental frequency, f_n , is 10.5 Hz, all modes up to 21 Hz should be considered. This effective impulse is used to calculate an initial velocity, \hat{v}_m , for each mode considered, which defines a time history of the decaying vibration response of a single mode, $v_m(t)$.

$$\hat{v}_m = \mu_{r,m} \mu_{e,m} \frac{I_{\text{eff},m}}{\hat{m}_m}$$

$$v_m(t) = \hat{v}_m e^{-2\pi\zeta f_m t} \sin(2\pi f_m t)$$

The time histories of the modes considered are then added together to get the total velocity response to one footfall, $v(t)$, and an RMS value for the total velocity is calculated. The duration of the time history used to calculate v_{RMS} should equal the time between footsteps, T_w , equal to $1/f_w$.

$$v(t) = \sum_{m=1}^N v_m(t)$$

$$v_{\text{RMS}} = \sqrt{\frac{1}{T_w} \int_0^{T_w} (v(t))^2 dt}$$

Discrete time increments, Δt , can be used to approximate the integral:

$$v_{RMS} = \sqrt{\frac{1}{T_w} \int_0^{T_w} (v(t))^2 dt} \cong \sqrt{\frac{1}{T_w} \sum_{i=0}^N (v(i * \Delta t))^2 \Delta t}$$

Where Δt is small enough so as not to alter the result.

A sample response history for walking at a single point on the floor is presented in Figure 4-6.

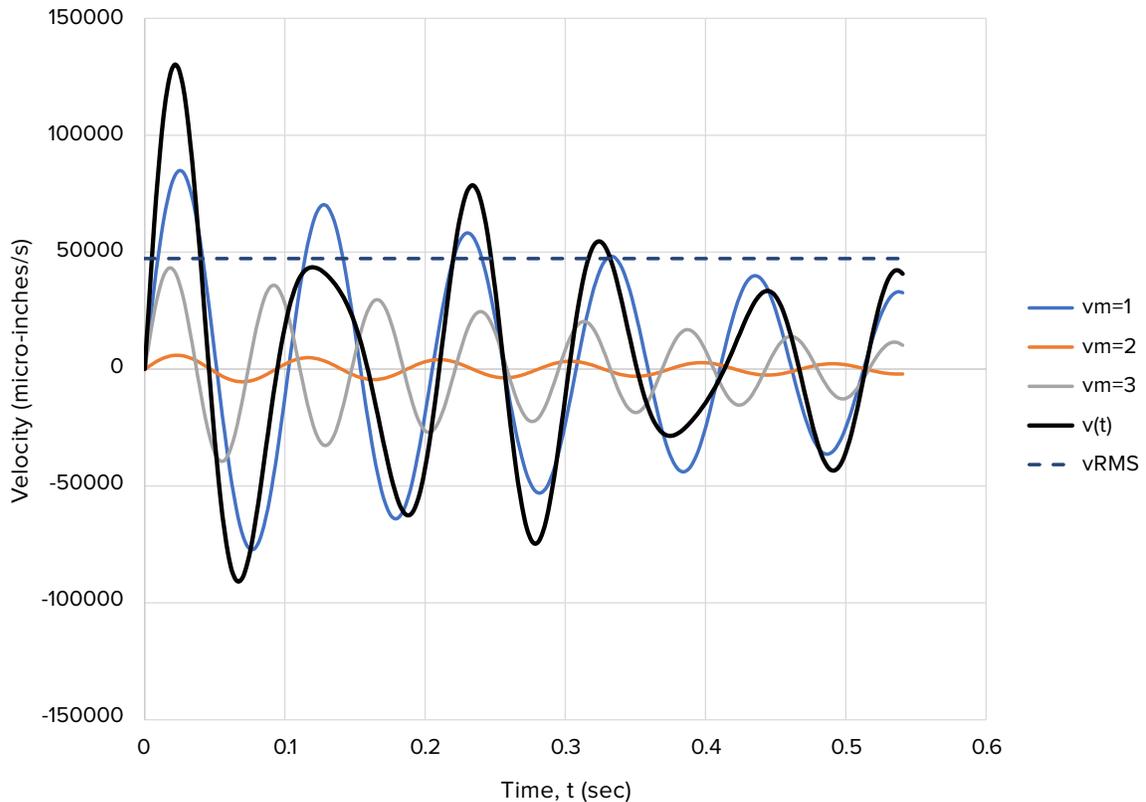


FIGURE 4-6: Sample velocity response history

This process is repeated for all points of interest, typically making use of a post processor as described in Chapter 6.

For human perception and velocity criteria VC-A and VC-B, a frequency weighting can be applied to v_{RMS} before comparing it with the selected performance target. The equation for frequency weighting is as follows:

$$v_{RMS, fw} = v_{RMS} \times \begin{cases} 1 & \text{if } f_1 \geq 8 \text{ Hz} \\ \frac{f_1}{8} & \text{if } f_1 < 8 \text{ Hz} \end{cases}$$

For floors with a fundamental frequency above 8.0 Hz, the frequency weighting does not change the calculated velocity, and v_{RMS} can be compared directly with the performance target selected.

In AISC Design Guide 11 and CCIP-016, the generic velocity criteria for sensitive equipment are defined using the RMS velocity measured within one-third octave bands, $v_{\text{RMS},1/3}$. When using the one-third octave band criteria, $v_{\text{RMS},1/3}$ is calculated at center frequencies with one-third an octave separation, e.g., 4, 5.04, 6.34, 8, 10.08 Hz, etc. A frequency is one-third an octave higher than a frequency multiplied by $2^{1/3}$. When calculating the $v_{\text{RMS},1/3}$ around a center frequency, instead of summing the modal velocity time history, $v_m(t)$, of all modes considered, only the modes with frequencies within the one-third octave band around the center frequency are included. For example, when calculating $v_{\text{RMS},1/3}$ at 8 Hz, the summation of $v_m(t)$ to find $v_{1/3}(t)$ includes only those modes with frequencies in the range of 7.13 Hz ($8 * 2^{-1/6}$) to 8.98 Hz ($8 * 2^{1/6}$). This process is repeated over the set of center frequencies. The largest value of $v_{\text{RMS},1/3}$ is the critical value compared to the selected performance target or velocity criteria.

4.4 Time History Analysis Method

The most precise assessment of floor response is a time history analysis incorporating an estimation of walking paths, forcing functions and response points on the floor at which vibration should be measured. Under this analysis, footfall loads will be applied along a prescribed walking path and the subsequent motion of the floor slab is measured at the excitation locations of interest. This level of analysis is beyond the scope of this guide and requires significant knowledge and professional judgment in the selection of appropriate parameters. More information on time history analysis can be found in the following guides, among other sources:

- AISC Design Guide 11
- SCI P354
- CCIP-016

Simplifying the Complexity of U.S. Customary Units

When working with dynamics equations with U.S. customary units, questions quickly arise about what units to use. Should one use pounds for mass (lbm) or force (lbf)? Do slugs need to be used for mass? In this guide, pounds are used for weight only (lbf). For any equation requiring a mass, recall $f = m * a$ and $w = m * g$. To find the mass given the weight:

$$m = w / g$$

where the calculated units of mass are lbf s²/in. In this guide, we keep mass in those units. One can convert the mass to slugs where 1 lbf s²/ft equals 1 slug, but this just adds unnecessary complexity for engineering applications where gravity is an assumed constant. Just remember $w = m * g$ and $m = w/g$ and use lbf throughout.

When using structural analysis packages, an understanding of the units and output options available to the user is needed. If you are unsure, double check the output of a simple analysis with hand calculations. While hard to double check by hand, pay close attention to the output units of modal analysis. The modal frequency is commonly Hz (cycles/second) but could be a radial frequency (rad/second). The modal mass and modal displacements may be normalized in many ways with varying units in the output.

TABLE 4-6: CLT Handbook method recommended span limit

Grade	t_p (in)	$E_{eff,f,0}$ (10^6 lbf-in ² /ft)	G	Density, ρ , at MC=12% (pcf)	$\bar{\rho}$	A (in ² /ft)	L _{lim} (ft)
E1	4.125	115	0.42	28.1	0.45	49.5	13.1
E1	6.875	440	0.42	28.1	0.45	82.5	18.2
E1	9.625	1,080	0.42	28.1	0.45	115.5	22.7
E2	4.125	102	0.49	32.5	0.52	49.5	12.4
E2	6.875	389	0.49	32.5	0.52	82.5	17.2
E2	9.625	963	0.49	32.5	0.52	115.5	21.6
E3	4.125	81	0.36	24.2	0.39	49.5	12.0
E3	6.875	311	0.36	24.2	0.39	82.5	16.7
E3	9.625	769	0.36	24.2	0.39	115.5	20.9
E4	4.125	115	0.55	36.3	0.58	49.5	12.7
E4	6.875	440	0.55	36.3	0.58	82.5	17.6
E4	9.625	1,089	0.55	36.3	0.58	115.5	22.1
E5	4.125	101	0.43	28.7	0.46	49.5	12.6
E5	6.875	389	0.43	28.7	0.46	82.5	17.5
E5	9.625	962	0.43	28.7	0.46	115.5	21.9
V1	4.125	108	0.49	32.5	0.52	49.5	12.6
V1	6.875	415	0.49	32.5	0.52	82.5	17.6
V1	9.625	1,027	0.49	32.5	0.52	115.5	22.0
V1(N)	4.125	108	0.49	32.5	0.52	49.5	12.6
V1(N)	6.875	415	0.49	32.5	0.52	82.5	17.6
V1(N)	9.625	1,027	0.49	32.5	0.52	115.5	22.0
V2	4.125	95	0.42	28.1	0.45	49.5	12.4
V2	6.875	363	0.42	28.1	0.45	82.5	17.2
V2	9.625	898	0.42	28.1	0.45	115.5	21.5
V3	4.125	95	0.55	36.3	0.58	49.5	12.0
V3	6.875	363	0.55	36.3	0.58	82.5	16.7
V3	9.625	899	0.55	36.3	0.58	115.5	20.9
V4	4.125	74	0.36	24.2	0.39	49.5	11.7
V4	6.875	285	0.36	24.2	0.39	82.5	16.3
V4	9.625	706	0.36	24.2	0.39	115.5	20.4
V5	4.125	88	0.43	28.7	0.46	49.5	12.1
V5	6.875	337	0.43	28.7	0.46	82.5	16.8
V5	9.625	835	0.43	28.7	0.46	115.5	21.0
S1	4.125	132	0.64	41.8	0.67	49.5	13.0
S1	6.875	506	0.64	41.8	0.67	82.5	18.1
S1	9.625	1,252	0.64	41.8	0.67	115.5	22.6
S2	4.125	114	0.64	41.8	0.67	49.5	12.4
S2	6.875	438	0.64	41.8	0.67	82.5	17.3
S2	9.625	1,085	0.64	41.8	0.67	115.5	21.7
S3	4.125	114	0.64	41.8	0.67	49.5	12.4
S3	6.875	438	0.64	41.8	0.67	82.5	17.3
S3	9.625	1,085	0.64	41.8	0.67	115.5	21.7

5

Modeling Approaches and Recommendations

5.1 Introduction

Modeling of mass timber floors for vibration evaluation can range from the use of simplified models for hand calculations to explicit modeling of an entire floor system. This section focuses mainly on considerations associated with detailed computer models. Figure 5-1 shows a representative example. The model for floor vibration primarily establishes the floor frequencies and mode shapes *for estimating vibration*. Recognizing this fact, models for floor vibration require different assumptions than those for strength or deflection criteria (e.g., connection fixity, composite action, representation of nonstructural elements, etc.); therefore, floor vibration models are typically separate from those used for evaluating strength or deflection requirements.

In vibration models, it is important to consider appropriate restraint and constraint conditions provided by both structural (e.g., column locations) and nonstructural (e.g., interior and exterior architectural walls) building elements, as well as treatment of connections and their relative rigidity/fixity. It is also important to provide accurate material properties of mass timber elements. Critical output results determined from a vibration model to be used for post-processing (Chapter 6) are the mode shapes, modal frequencies and modal masses. Greater detail on these topics is provided in the following subsections.

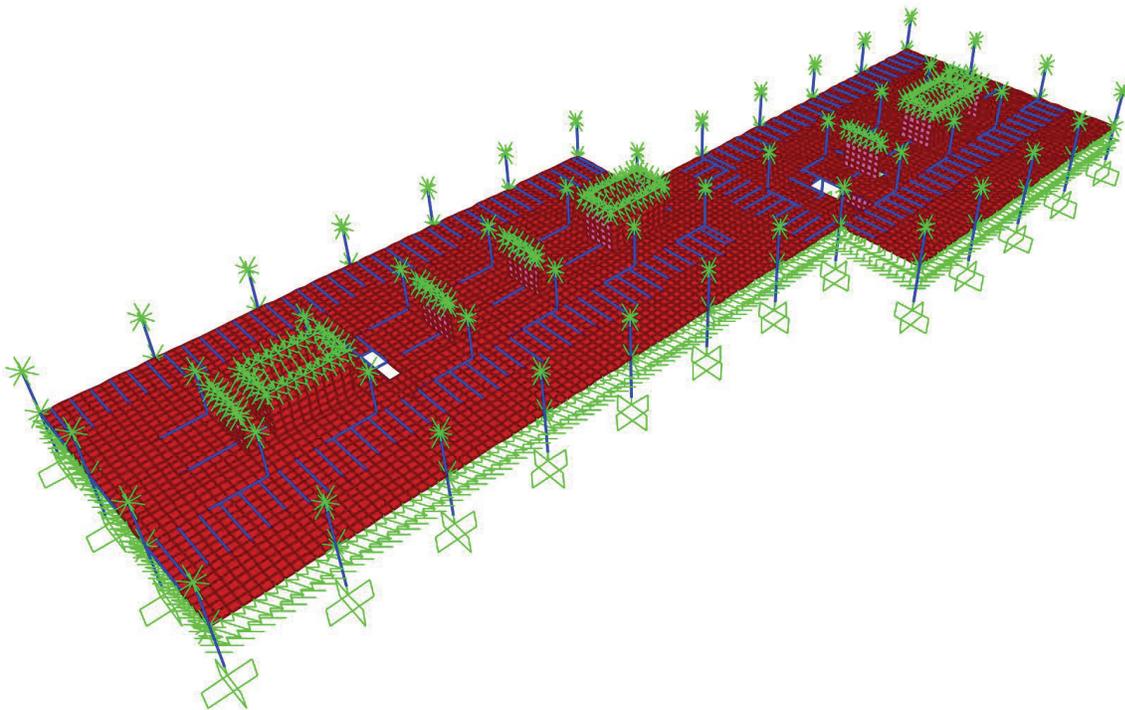


FIGURE 5-1: Example floor vibration model in SAP2000

5.2 Basic Elements and Features in Finite Element Models

A computer model of a floor used to estimate vibration will employ finite element analysis. The following basic elements and features can be found in essentially every finite element software. They are introduced here, along with typical mass timber applications, to define the nomenclature used in later sections of this guide.

- **Joint elements** are zero-dimensional elements comprising a single (X, Y, Z) position in space. They are typically used to define the connectivity of distinct frame or shell elements, or as a basis on which restraints or constraints are applied. However, joint elements are also useful for modeling partial fixity between frame or shell elements (e.g., the flexibility of a beam-to-column connection modeled as two intersecting frame elements at a common joint element).
- **Frame elements** are one-dimensional elements comprising a position in space between two (X, Y, Z) points often represented with joint elements. Frame elements are useful for modeling structural components with properties that are constant (or can be discretized to be constant) along a single line. Joists, beams, girders and columns are examples.
- **Shell elements** are two-dimensional elements comprising a position in space defined by a plane (i.e., at least three (X, Y, Z) points). Shell elements are useful for modeling structural components that are planar, such as walls, decks and floor panels. Two special classes of shell elements are plate and membrane elements. **Plate elements** have only out-of-plane bending/shear and no in-plane properties. **Membrane elements** have only in-plane bending/shear and no out-of-plane properties. A shell element has both plate and membrane properties.
- **Solid elements** are three-dimensional elements. They are rarely used in floor vibration models.
- **Restraints** are used to represent the boundary between what is and is not modeled. Restraints are fixity in any or all of the six degrees of freedom (i.e., translation in and rotation about each X, Y and Z axis). Partial-fixity restraints (i.e., a restraint that is not perfectly rigid) can be implemented in some finite element software through either a restraint or joint element properties or both.
- **Constraints** are rules that relate degrees of freedom of one joint element to another. For example, the commonly used rigid body constraint specifies that all joint elements assigned to the constraint must move together as if connected by a rigid body. Constraints can be useful when the structural components connecting points in space are much, much stiffer than the surrounding components or when three-dimensional connectivity is required between elements modeled only using one-dimensional (frame) or two-dimensional (shell) elements. For the example of a floor slab that is fully composite with the beam below and where each is modeled with elements about the respective centerlines, a rigid body constraint can be used to connect the joint in the slab with the adjacent joint in the beam separated by one-half the beam depth plus one-half the slab thickness.
- **Internal-to-element releases** allow force actions (e.g., shear or moment) to be released at the end/edge or internal to the element. This feature is provided in most finite element software for ease of use in lieu of dividing the element and assigning similar properties to a joint element at the intersection.

5.3 Model Extent, Geometry, Restraint and Mass Modeling

The first decision the analyst must make when building a model for estimating floor vibration is the spatial extent of the model. Generally floors in multi-story buildings are modeled independently for vibration estimation. However, there are special cases where multiple floors within a single model may be necessary, as discussed later. In the more typical case, the analyst must decide whether the entire floor or only a portion should be modeled.

It is recommended that the entire floor be modeled to accurately capture continuity provided by adjacent spans, and to avoid simplifying assumptions at the model boundary, thus overly influencing results. Other guidance documents such as AISC Design Guide 11 recommend that modeling, at a minimum, extend one bay in each direction beyond the bay in question. If only a segment of the full floor is modeled, it can be valuable to include sensitivity studies on the boundary conditions representing the remaining floor not included in the model.

The model should include the structural, and sometimes nonstructural, components that contribute to the stiffness of the floor. The associated decisions about connection modeling and composite action (Sections 5.5 and 5.6) should also be made at this stage.

Application of mass to the floor vibration model should represent the distribution expected in the constructed building and include likely, not worst case, dead loads and some proportion of the live load. Likely dead and live loads are almost always less than those specified for design for strength and deflection. See Section 3.1 for information on estimating likely live loads. Since floor vibration models are typically built to represent the relevant vertical, rather than horizontal, dynamic properties, it is common to only assign vertical mass if that option is available in the modeling software selected.

Instead of modeling the entire structure, a floor vibration model will typically include a single level with restraints at the boundary of the floor plate to represent portions of the structure not included in the model. For example, it is common to model a half-story height of columns and walls above and below the modeled floor to approximate the equivalent rotational stiffness provided at the column or wall location. At the mid-story height, a pinned restraint (i.e., three translational degrees of freedom fixed, three rotational degrees of freedom free) is often used to approximate the displacement and force compatibility with the rest of the non-modeled structure. To provide lateral stability to the model, horizontal translation restraints should be located at the tops of columns (Figure 5-2). These modeling recommendations assume columns are supported directly to foundations; modeling of columns supported by a transfer level may need to be modified to account for the flexibility and mass of the supporting framing.

For computer models to represent in-situ tested floor response, both interior and exterior full-height nonstructural/non-load-bearing walls need to be represented as vertical springs (Figure 5-2). This is true even when the connection details allow for vertical movement between the floor and walls (e.g., a nested deflection track in light-gauge walls) since the forces associated with floor vibration are not sufficient to initiate slip in these details. AISC Design Guide 11 recommends vertical springs with a stiffness of 2.0 kip/in. per ft of wall length for nonstructural/non-load-bearing walls that frame from top of floor to bottom of structure. Partitions that do not extend to the underside of the framing above (e.g., nonstructural walls that stop just above a dropped ceiling with light-gauge or light-frame kicker bracing) likely do not warrant modeling with restraints or springs.

Whether the partitions will be present for the life of the structure is crucial when considering the inclusion of nonstructural walls in a model. In residential construction, interior partitions are generally fixed for the life of the building, whereas in office construction, partitions will be rearranged many times. Unless they are expected to be permanent, relying on a specific arrangement of partitions is inappropriate for a floor vibration model. Where interior partitions are impermanent or their exact locations are not known, the effect of partitions can instead be incorporated through additional damping (Section 3.2). It should be noted that representing partitions through additional damping rather than directly through restraints or springs is moderately to significantly conservative for vibration evaluation.

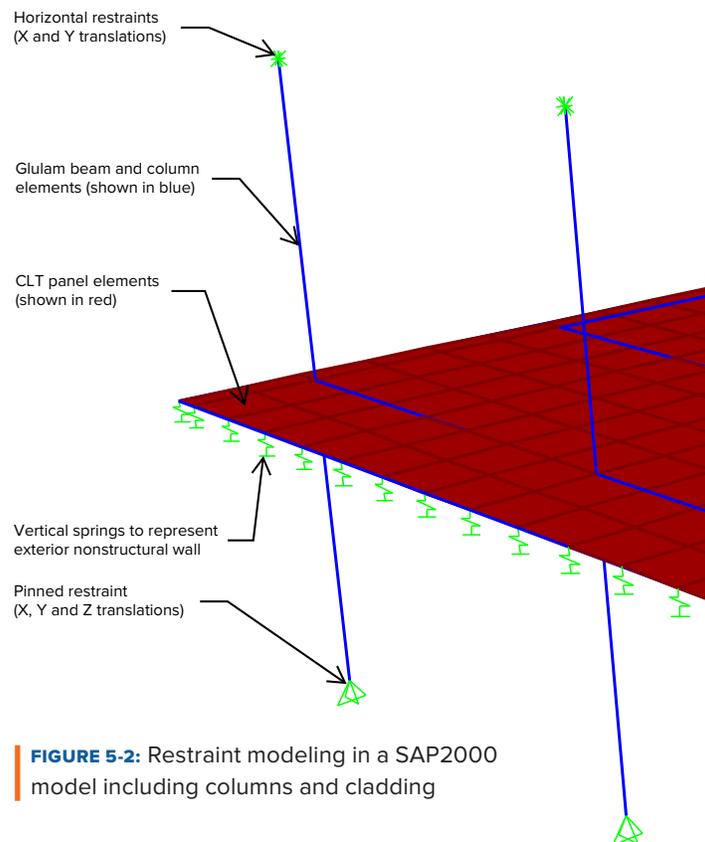


FIGURE 5-2: Restraint modeling in a SAP2000 model including columns and cladding

5.4 Material Property Modeling

Mass timber floors have anisotropic properties which, as described in Section 3.3, can be modeled in several ways, including:

1. Isotropic material with section property modification factors
2. Orthotropic material
3. Layered laminate sections

A simplified approach to defining the material properties of shell elements is to define an isotropic material/section and then apply different property modification factors by direction. This method is available in many structural analysis platforms. Figure 5-3 shows the local axes and corresponding stress orientation definitions for shell elements in one commercially available software platform (SAP2000). It will be referenced to illustrate the development of property modification factors, but the procedure can be used in any software that supports property modification factors. Assume that Axis 1 in Figure 5-3 aligns with the strong axis of a mass timber panel (i.e., the direction of the panel's primary span). The effective stiffness properties from Section 3.3 for the mass timber panel can then be divided by the product of the reference isotropic material property and gross section property to determine the respective property modification factor (Table 5-1). Chapter 9 provides more detail on software-specific implementation of this general procedure.

TABLE 5-1: Property modification factors for isotropic material modeling

Orientation	Property Modification Factor
Strong axis axial	$f_{11} = \frac{EA_{eff,0}}{E_{ref} * A_{gross}}$
Weak axis axial	$f_{22} = \frac{EA_{eff,90}}{E_{ref} * A_{gross}}$
In-plane shear	$f_{12} = \frac{GA_{eff,e}}{G_{ref} * A_{gross}}$
Strong axis flexure	$m_{11} = \frac{EI_{eff,f,0}}{E_{ref} * I_{gross}}$
Weak axis flexure	$m_{22} = \frac{EI_{eff,f,90}}{E_{ref} * I_{gross}}$
Out-of-plane torsion*	$m_{12} = \min(v_{13}, v_{23})$
Strong axis out-of-plane shear	$v_{13} = \frac{GA_{eff,f,0}}{G_{ref} * A_{gross}}$
Weak axis out-of-plane shear	$v_{23} = \frac{GA_{eff,f,90}}{G_{ref} * A_{gross}}$

* Consensus on the out-of-plane torsion stiffness of most mass timber products does not exist in the literature. Its property modification factor is therefore conservatively taken as the lower of that for strong or weak axis out-of-plane shear.

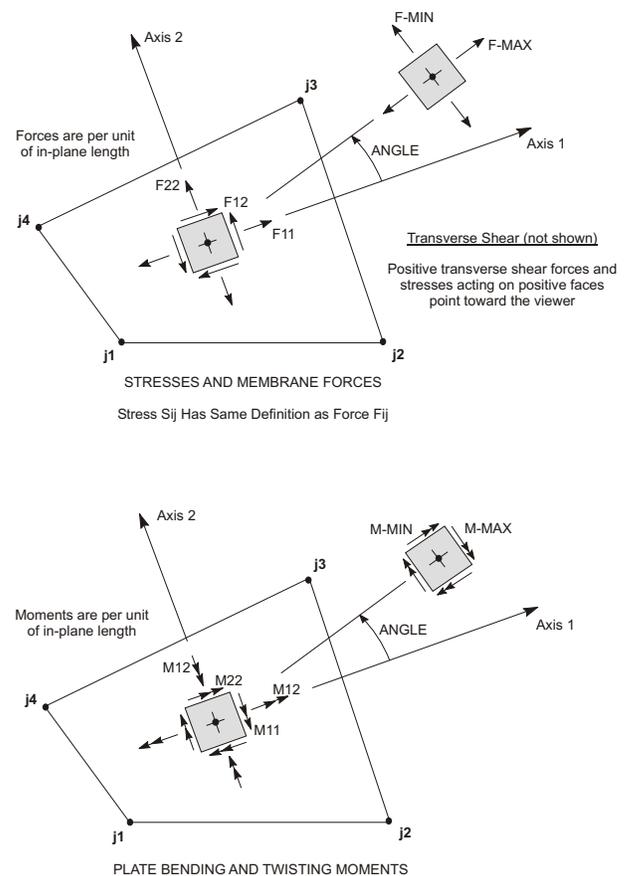


FIGURE 5-3: Local axes and corresponding stress orientation definitions for one commercially available software platform (SAP2000)

Use of an orthotropic material with or without layered laminate sections allows more direct specification of effective properties generally without the need for property modification factors. This approach can relieve the analyst from manually calculating effective properties, but can sometimes mask internal software assumptions that may not be appropriate for vibration evaluation. Additionally, orthotropic materials used for representing mass timber members with a large difference in effective properties between directions (e.g., tongue-and-groove decking where $EA_{\text{eff},90} \approx 1/30 * EA_{\text{eff},0}$) can present numerical instabilities when using orthotropic material definitions.

5.5 Connection Modeling

Vibration evaluation can be relatively sensitive to connection modeling and it is recommended that the analyst not be overly conservative when modeling connections. Vibration due to walking excitation is a result of low-level force variation in the structure (i.e., the impact force of the walker's step) as compared to dead and live loads in building code load combinations. While a certain connection may respond most like an idealized pin (i.e., moment release) for strength and deflection evaluation, that same connection often responds most like a fully restrained connection for vibration cases. Typical connections in mass timber or glulam post-and-beam structures and recommended modeling approaches include:

- **Beam-to-column or beam-to-girder** – Typical types of glulam beam-to-column and beam-to-girder connections include bearing/seated, hanger and proprietary concealed. In almost all cases, these can be considered fully restrained for a vibration model. In special and uncommon cases (e.g., top flange hung beam with a gap between the beam end and girder face), either a joint spring or full moment release may be appropriate in lieu of fully restrained modeling.
- **Column-to-column splice** – Typical types of glulam column-to-column splice connections include direct end-to-end bearing of the glulam columns via a steel plate or a short section of steel pipe through mass timber floor panels to avoid compression perpendicular-to-grain loading. Similar to discussion of beam-to-column and beam-to-girder connections, glulam column-to-column splices can almost always be considered fully restrained for vibration evaluation.
- **Floor panel-to-wall panel** – Typical mass timber floor panel-to-wall panel connections include direct bearing (i.e., the floor panel bears directly on top of or between wall panels) or a ledger connection (i.e., a steel angle or wood block attached to the face of the wall panel on which the floor panel rests). It is recommended that mass timber floor panel-to-wall panel connections using direct bearing be modeled with a joint spring or full moment release unless it can be demonstrated that the clamping force provided by the platform-framed wall above is sufficient to create full flexural continuity through the floor panel-to-wall panel connection. In a ledger condition, a joint spring or a full moment release is also appropriate.
- **Floor panel-to-panel splice** – Typical mass timber floor panel-to-panel splices include surface spline and half-lap connections. Both provide some continuity of the floor panels through the splice, although this stiffness is less than the adjacent floor panels. Explicit modeling approaches include using strips of shell elements or line springs at the splice location with properties that represent the flexural and axial flexibility of the splice. As a simplifying, conservative approach, the splices may instead be modeled with moment and axial releases. It is acknowledged that floor panel layout may not be determined until the beginning of construction (and therefore cannot be considered explicitly in the model) and that estimation of splice stiffness, which is dependent on the number and type of fasteners, is not straightforward. The analyst may therefore need to assess the sensitivity of the vibration evaluation results to the location and stiffness of floor panel-to-panel splices. Furthermore, the flexural and axial stiffness of a concrete topping also contribute across a floor panel-to-panel splice and may be included. However, the axial stiffness of the topping should be neglected unless composite action is expected (Section 5.6).

Note that the analyst may choose to represent vertical structural components (e.g., columns and walls) with restraints when defining the model extent and geometry rather than fully modeling the component a half-story height above and below. In this case, it would be unconservative to include a fully fixed restraint at the location of the column or wall since the flexibility of the column or wall is neglected. Instead, a vertical translation (i.e., pinned) restraint only is more appropriate and conservative. The conservatism of a pinned restraint could be mitigated by either modeling the vertical structural components (i.e., walls or columns) or by representing the column or wall's flexural stiffness as a rotational spring calibrated accordingly.

5.6 Composite Action Modeling

The potential for composite action exists in mass timber floors at two primary interfaces to be considered in a vibration model: (a) interface between a glulam beam/girder and a mass timber floor panel, and (b) interface between a mass timber floor panel and a concrete topping. Modeling the composite action between a beam and floor panel is accomplished in one of two ways as shown diagrammatically in Figure 5-4:

- **Implicit modeling approach** – A frame element is used to represent the beam (modified for composite action) in conjunction with shell elements for the mass timber panel. All elements are modeled in the same horizontal plane. Alternatively, the stiffness of the glulam beam/girder may be distributed into the effective properties of the shell element representing the mass timber panel.

- **Explicit modeling approach** – The glulam beam/girder and the mass timber floor panel are modeled with distinct frame and shell elements, respectively, about their respective centerlines with a constraint or spring modeled between them.

The explicit modeling approach is most conducive when fully composite action is assumed between the beam/girder and floor panel and a rigid offset/constraint can be used. When partially composite action needs to be considered, explicit modeling can make it challenging to determine the required joint spring stiffness (unless derived from testing at low force levels). While the implicit modeling approach may therefore be preferred, it then requires estimation of the appropriate percent composite action factor. For recommendations concerning composite action between glulam beams/girders and mass timber floors, refer to Section 3.4. It should be noted that implicit and explicit modeling can be used in combination in the same model. For example, a CLT floor with composite concrete topping may be modeled implicitly while the glulam beam-to-CLT panel connection may be modeled explicitly.

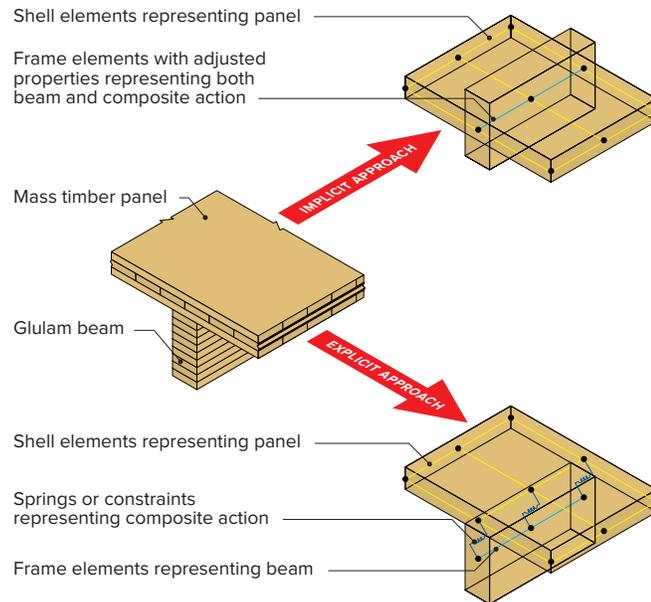


FIGURE 5-4: Implicit vs. explicit modeling of a glulam beam and a mass timber floor panel

Where utilized, a gypcrete or concrete topping must be included in the vibration model through added mass. However, concrete toppings also provide additional stiffness and, in some cases, may be considered to act partially or fully composite with the mass timber floor panels and beams/girders below. This may be true even when the concrete topping is not specifically designed to be composite for strength or deflection. Composite action between mass timber panels and concrete toppings is most easily modeled via an implicit modeling approach, where shell elements representing both the mass timber floor panels and topping slab are assigned effective stiffness properties (Section 3.4.2). The shell elements are then modeled at the effective section centroid. Even when the recommended percent composite action is zero ($\gamma_1 = \gamma_2 = 0$ in Section 3.4.2), the flexural stiffness of the topping itself still contributes to the combined flexural stiffness.

5.7 Defining a Modal Analysis

Most finite element software is set up to perform a modal analysis with default parameters (i.e., with little direct input by the analyst). The most common type of modal analysis for structural engineering applications uses the Eigen modes as its basis. While other options are available (e.g., Ritz modes) and can be used in floor vibration estimation, Eigen modes are recommended since they are more easily interpreted by the structural engineering community.

The number of modes needed depends on the size of the model and eventual use of the modal data. Even if only the fundamental frequency and mode shape are sought for use in a simplified method, it is recommended that multiple modes be calculated so the analyst can review which is appropriate. This is especially true for a large floor model to ensure the floor area of interest is captured in the mode shapes. See Section 6.3 for more information on selecting the appropriate mode for simple calculations. When a modal response analysis method such as the CCIP-016 procedure is implemented through a post-processor (Section 6.4), the minimum number of modes is often prescribed, as discussed in Section 4.3.

5.8 Meshing for Post-Processing

Chapter 6 describes how a vibration model is used in conjunction with a post-processor to perform a modal response analysis method. Mesh size and corresponding spacing of joints are key aspects of interfacing between a model and the post-processor, including:

1. Where possible, have an even number of elements per bay so a joint exists at the center of a bay; an odd number of elements per bay would result in the geometric center of the bay falling at the center of an element rather than a joint. For irregularly shaped bays, it is recommended to locate a meshing joint at the center of a bay for post-processing ease.
2. The mesh must be fine enough that two adjacent joints are not more than approximately 3 ft apart. If the mesh exceeds 3 ft, the post-processor may not be able to assess the effect of an excitation location near, but not too near, a response location. See Section 6.4 for additional discussion.

5.9 Experimental Calibration

FEA models are built upon numerous decisions regarding stiffness, boundary conditions and analysis techniques. As such, it is valuable to calibrate models and modeling techniques with measurements taken from physical tests. Such tests can be from similar components, assemblies, or built projects. In the design of large buildings, or the development of floor framing systems to be repeatedly built, an investment in building and testing prototypes or mock-ups of the floor system can be very beneficial. Such tests can be used to fine-tune the modeling approaches and avoid problematic or overdesigned floors.

6

Model Results Interpretation and Post-Processing

6.1 Modeling Implementation Checks

Before using data from the modal analysis of the floor vibration model, it is prudent to perform several modeling implementation checks to ensure that (a) modeling choices and assumptions, and (b) oversights or errors in modeling or property definitions have not led to unrealistic results. Checks include:

1. Most finite element software platforms keep a log of errors and warnings if present during the analysis. These should be checked regularly.
2. Review modal frequencies for frequencies of the wrong magnitude (e.g., 0.2 Hz or 50 Hz). Estimated modal frequencies should compare to simple hand calculations or rules of thumb. See Section 4.2 for suggested simplified methods.
3. If the software allows, it is helpful to review still graphics, or ideally animations, of the mode shapes to look for disconnected joints (e.g., errors in modeling geometry/connectivity) or unexpected/odd mode shapes (e.g., modes that lack translational or rotational continuity between elements expected to be connected).
4. Similar to visualization of mode shapes, displaying the deformed shape under dead and/or live load can be useful in debugging a model.
5. If the software allows, reviewing the model with 3D extrusions and element offsets shown will help diagnose errors in property definitions, local axis orientations and element offsets.
6. Check that base reactions sum to the expected input forces.
7. Finally, the modal mass for the mode with greatest contribution at the location of interest (e.g., center of a bay) should be compared to the mass of a single bay. When the modal mass significantly exceeds the mass of a single bay, the model should be reviewed carefully. In some cases, such as a floor with regular bay sizes and framing, it is possible for a single mode to involve a significant portion of the floor. While this is analytically correct, slight variation in properties and fixity in the actual building will likely break that high modal mass mode into several, closely spaced lower modal mass modes. In such a situation, the analyst should consider this reality through methods such as random property modification or restricting the extent of modeling to less than the full floor.

6.2 Initial Results Screening

Once the initial design assumptions are made, and the model is operating consistent with those assumptions, the results should be reviewed at a high level for signs of potential poor vibration performance. This review can be done qualitatively before performing calculations or detailed post-processing. The review process may suggest design changes that should be implemented before committing additional time evaluating a design that is not likely to satisfy the project's vibration performance targets. Table 6-1 provides options for high-level review and how to address common issues.

TABLE 6-1: Initial results screening

What to Check	Why Check	Action to Take
Modal frequencies less than 6 Hz	Modal frequencies in this range often lead to poor vibration performance.	<ul style="list-style-type: none"> • Review model to eliminate overly conservative assumptions. • Increase stiffness (or decrease mass) in the structural components contributing to this mode (e.g., by activating composite action between structural components). • Move vibration-sensitive occupancies away from the area (often difficult or unacceptable to the rest of the design team). • Perform more advanced analysis to achieve a quantitative measure of performance.
Modes with mode shapes simultaneously active at both: <ul style="list-style-type: none"> • Sensitive occupancy locations (e.g., classroom or laboratory) • Stronger vibration input locations (e.g., corridor) 	Sensitive occupancies adjacent to strong input occupancies are subject to high vibration input (i.e., fast walking speeds), which often leads to poor vibration performance.	<ul style="list-style-type: none"> • Break the dominant mode, potentially into two distinct modes, one for each area, by adjusting the floor design so vibration from the high-input area is not as easily transmitted to the sensitive area. • Move sensitive areas away from high-input areas (often difficult or unacceptable to the rest of the design team). • Perform more advanced analysis to achieve a quantitative measure of performance.

6.3 Finding the Appropriate Mode for Simplified Procedures

A vibration model is typically developed for use with the modal response analysis method (Section 4.3). Such a model may also be developed simply to determine the natural frequency and corresponding modal mass for use in a simplified procedure. It is important to understand how different software platforms treat normalization of modes when attempting to determine the mode to use in simplified procedures. For example, some software platforms normalize each mode such that the maximum mode shape ordinate is 1.0, where others normalize each mode such that the modal mass is 1.0 for a given set of units. In the latter case, the mode to use for a given location is the mode with the largest mode shape value at that location. However, in the former case, it is necessary to find the largest of the quantity equal to the mode shape value squared divided by the modal mass. If the mode to use for a given location is not readily apparent when viewing the modal results, or if several modes appear to be dominant, simplified methods may not be appropriate and a modal response analysis method is likely warranted.

6.4 Implementing a Post-Processor

When implementing a modal response analysis method, it is useful to create a post-processor to handle the relatively large amount of data and to perform the detailed calculations presented in Section 4.3. This can be done with a range of calculation tools as accessible as Microsoft Excel or as sophisticated as a C++ script. The following model outputs must be exported from the floor vibration model and imported into the post-processor for any relevant modes:

- Modal frequency
- Mode shape values at the excitation and response nodes
- Modal mass

It is essential that the units of the model and post-processor are aligned (e.g., lbs-in., kips-ft, etc.), especially since many of the modal response analysis method equations are unit dependent.

The post-processor will also include these user-defined inputs:

- f_w – Walking frequencies of interest, with a range of low to high (Section 3.6.1)
- P – Static weight of the walker (Section 3.6.2)
- l – Stride length of the walker (Section 3.6.3)
- L – Span length of bay in which excitation node resides
- ζ – Damping as ratio to critical damping (Section 3.2)
- Response node – Joint at which vibration performance is to be evaluated
- Excitation node – Joint at which excitation is to be applied
- Performance criteria – Response criteria for the response node location (Section 3.7)

Note that the response and excitation node need not be taken closer together than 3 ft, since this would otherwise simulate the conservative self-excitation case discussed in Section 4.3. With both the user inputs and model outputs, the post-processor needs to complete the calculations outlined in Section 4.3. See Figure 6-1 for a screenshot of an example Microsoft Excel post-processor implementing the resonant response calculations outlined in Section 4.3.1 for the first ten modes.

	0.8 Hz	2.8 Hz	32.2 ft/sec ²	0.00257 g	h	1	h	2	h	3	h	4
$f_{w,min}$	0.8	2.8										
$f_{w,max}$												
g												
a_{total}												

	1	2	3	4
h	1	2	3	4
f_n	2	4	6	8
DLF	0.431	0.091	0.071	0.065
F_n	72.3	15.4	12.0	10.9
N	550	1100	1650	2200
ρ_n	1	1	1	1
a_n	0.0000527	0.000056	0.00002	0.003
$a_{n-1,h}$	0.00102	0.00072	0.00072	0.00072

Mode	f_m	$H_{2,m}$	$H_{4,m}$	$m_{m,hat}$	A_m	B_m	$a_{total,m}$	$a_{mag,h,m}$	A_m	B_m	$a_{total,m}$	$a_{mag,h,m}$	A_m	B_m	$a_{total,m}$	$a_{mag,h,m}$	A_m	B_m	$a_{total,m}$	$a_{mag,h,m}$
-	Hz	ft	ft	lb-ft-sec ²	-	-	ft/sec ²	ft/sec ²	-	-	ft/sec ²	ft/sec ²	-	-	ft/sec ²	ft/sec ²	-	-	ft/sec ²	ft/sec ²
1	6.69	-0.00017	-0.00017	83.33	0.91068	0.01195	2.32E-09	3.04E-11	0.64272	0.02291	2.78E-09	1.04E-10	0.19612	0.03586	1.55E-08	2.84E-09	-0.42912	0.04782	-1.17E-08	1.31E-09
2	7.57	0.00074	0.00074	83.33	0.93016	0.01057	3.53E-08	4.01E-10	0.72066	0.02114	3.87E-08	1.13E-09	0.37148	0.03171	1.31E-07	1.12E-08	-0.11737	0.04228	-5.98E-07	2.15E-07
3	7.99	-0.00044	-0.00044	83.33	0.93740	0.01001	1.13E-08	1.21E-10	0.74962	0.02002	1.20E-08	3.21E-10	0.43664	0.03002	3.61E-08	2.48E-09	-0.00152	0.04003	-2.43E-08	6.40E-07
4	8.00	-0.00576	-0.00576	83.33	0.93750	0.01000	1.92E-06	2.05E-08	0.75000	0.02000	2.04E-06	5.43E-08	0.43749	0.03000	6.11E-06	4.19E-07	-0.00002	0.04000	-4.35E-08	1.09E-04
5	8.00	0.03616	0.03616	83.33	0.93756	0.00999	7.55E-05	8.05E-07	0.75026	0.01999	8.01E-05	2.13E-06	0.43808	0.02998	2.40E-04	1.64E-05	0.00102	0.03998	1.10E-04	4.28E-03
6	8.02	0.09111	0.09111	83.33	0.93779	0.00998	4.78E-04	5.08E-06	0.75117	0.01995	5.06E-04	1.34E-05	0.44012	0.02993	1.51E-03	1.03E-04	0.00467	0.03991	3.13E-03	2.68E-02
7	8.10	0.04037	0.04037	83.33	0.93898	0.00988	9.19E-05	9.67E-07	0.75590	0.01976	9.69E-05	2.53E-06	0.45079	0.02964	2.85E-04	1.87E-05	0.02362	0.03952	2.32E-03	3.89E-03
8	8.11	0.00025	0.00025	83.33	0.93922	0.00986	3.62E-09	3.80E-11	0.75686	0.01972	3.82E-09	9.95E-11	0.45294	0.02959	1.12E-08	7.29E-10	0.02746	0.03945	9.77E-08	1.40E-07
9	8.11	-0.13679	-0.13679	83.33	0.93925	0.00986	1.05E-03	1.10E-05	0.75700	0.01972	1.11E-03	2.88E-05	0.45325	0.02958	3.24E-03	2.11E-04	0.02799	0.03944	2.85E-02	4.02E-02
10	8.13	-0.00244	-0.00244	83.33	0.93951	0.00984	3.33E-07	3.48E-09	0.75805	0.01968	3.50E-07	9.08E-09	0.45562	0.02951	1.02E-06	6.60E-08	0.03222	0.03935	9.40E-06	1.15E-05

FIGURE 6-1: Example post-processor resonant response calculations

A post-processor that implements a modal response analysis method such as the CCIP-016 procedure can produce graphs such as those shown in Figure 6-2 through 6-4 to aid the analyst in assessing acceptable vibration performance. Figure 6-2 shows the peak acceleration due to resonant response for a given excitation and response node. The total response is the square-root-sum-of-the-squares (SRSS) combination of the individual harmonics and would be the parameter used to assess the response against the selected performance criteria. For example, the current design shown in Figure 6-2 does not meet a specified limit of 0.5% g. Plotting of the individual harmonics is useful since it guides the analyst in finding the dominant mode. For example, the total response peak at approximately 2.1 Hz walking frequency in Figure 6-2 is almost completely composed of the fourth harmonic. Therefore, the dominant mode will have a frequency of approximately 8.4 Hz (= 2.1 Hz * 4). The analyst can then review the modal results to better understand that mode.

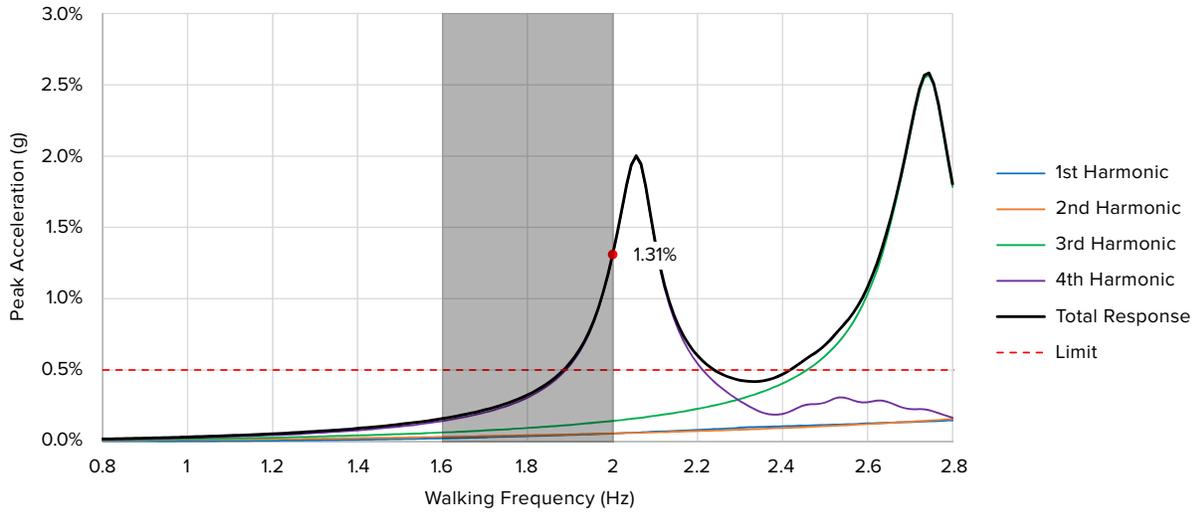


FIGURE 6-2: Example post-processor output for evaluating floor vibration performance based on resonant response for a given excitation and response node; area in grey indicates walking frequencies of interest

Figure 6-3 shows the RMS velocity due to impulsive response for a given excitation and response node, which can be used to assess the response against the selected performance criteria. For example, the current design shown in Figure 6-3 meets a specified limit of 32,000 micro-in./s. As described in Section 4.3.2, it is only necessary to evaluate the impulsive response for the fastest walking frequency expected. However, it is useful to include a range of walking frequencies in the post-processor setup to be able to assess sensitivity of floor vibration acceptability to walking frequency.

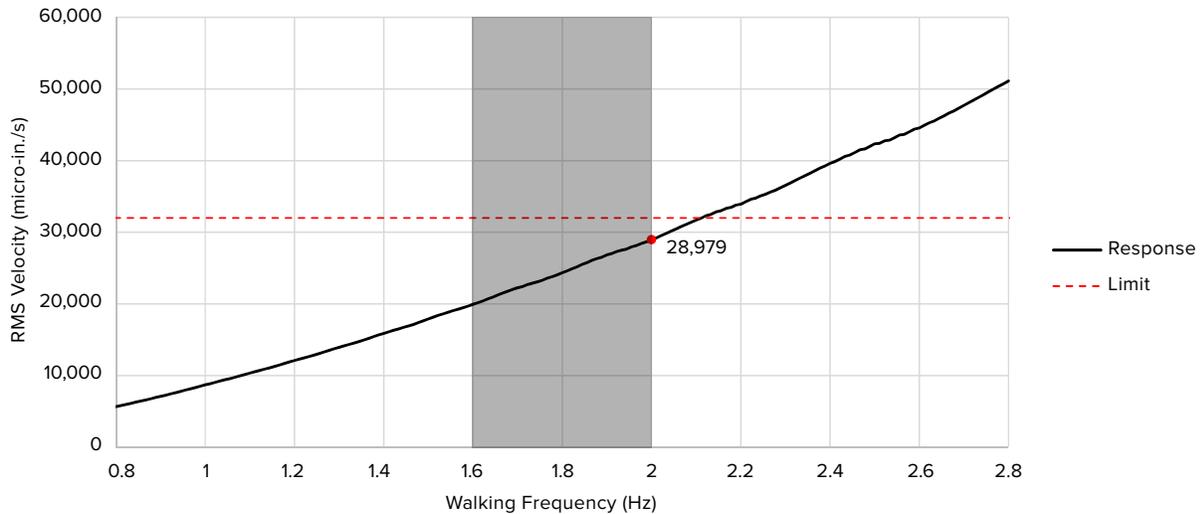


FIGURE 6-3: Example post-processor output for evaluating floor vibration performance based on impulsive response for a given excitation and response node; area in grey indicates walking frequencies of interest

Figure 6-4 shows a heat map illustrating an envelope of the results for all combinations of response and excitation nodes. It essentially performs the same calculations shown in Figure 6-2 and 6-3, but for all nodes in the model. While visually instructive, utilizing a post-processor that creates a diagram similar to Figure 6-4 requires much more computational time than one that produces results such as those in Figures 6-2 and 6-3.

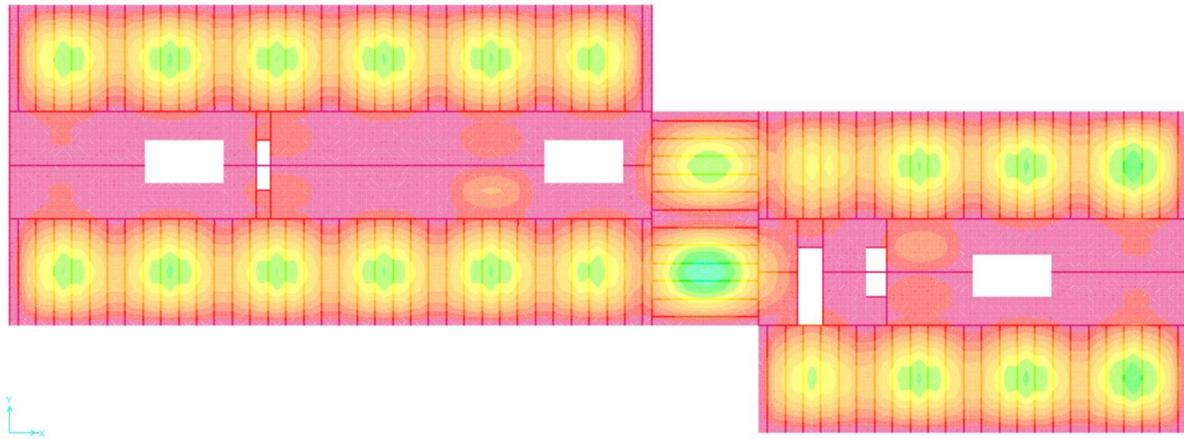


FIGURE 6-4: Example post-processor output for evaluating floor vibration performance based on an envelope of results for all combinations of response and excitation nodes; cooler colors indicate areas of higher (less desirable) floor vibration

7

Example 1: Bearing Wall Example Using RFEM

In this example, the vibration design of a continuous slab supported by bearing walls is considered. Side-by-side units 30-ft-long by 15-ft-wide are assumed (total floor plate under consideration is 30x30 ft), representing a typical multi-family residential building (Figure 7-1). The system consists of CLT bearing walls supporting three 10-ft-wide CLT floor panels. A 1.5-in. nonstructural concrete topping and dropped ceiling are included in the floor assembly (Figure 7-2). To study the impact of floor panel continuity, two models are considered, comparing isolated simple-span panels and two-span continuous panels (Figure 7-3).

The floors are first evaluated using simplified design tools and then using modal analysis methods discussed in Chapter 4. The software suite RFEM is used to model the floor and determine its modal properties; an Excel spreadsheet is then used to post-process the results and evaluate the performance.

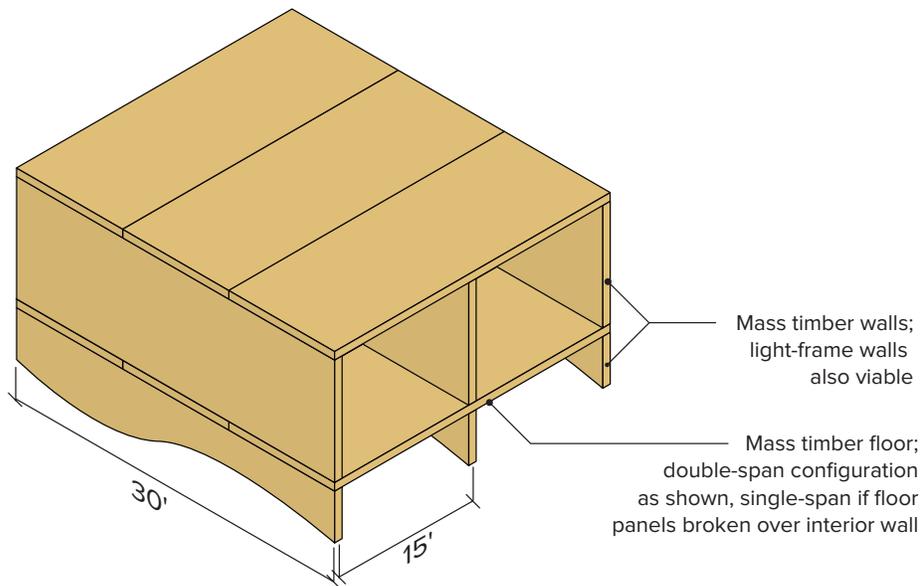


FIGURE 7-1: Floor system considered in this example

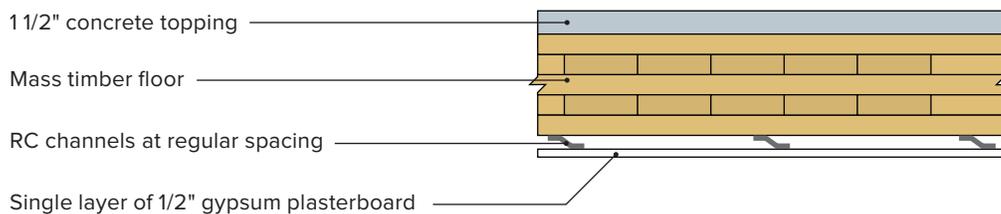


FIGURE 7-2: CLT floor assembly

7.1 System Property Definitions

As discussed in Chapter 3, it is important to accurately define the system properties for either hand calculations or more detailed modal response analysis. The properties for the system are described in the following sub-sections.

7.1.1 Component Stiffness

For model-based dynamic calculations, the floor stiffness is based on material properties for both the CLT and concrete topping. This example uses basic CLT grade V2 with a 6 7/8-in., 5-ply layup defined in the PRG 320-19 standard. The stiffness of the concrete topping is included without composite action for vibration analysis, as it is cast over an acoustic mat. The concrete topping properties are based on specified compressive strength of 4 ksi; elastic modulus is calculated according to ACI 318-19 (2019) and multiplied by 1.35 per Section 3.3.5 to account for the low-strain dynamic modulus of elasticity. The CLT panel and concrete topping layer properties are shown in Table 7-1. The walls supporting the floor panels are assumed to be rigid and are discussed in more detail in Section 7.1.4.

TABLE 7-1: Floor system properties including topping (per ft width)

Floor Layer	Thickness (in.)	Weight (psf)	$E_{\text{eff},f,0}$ (10^6 lbf-in. ² /ft)	$E_{\text{eff},f,90}$ (10^6 lbf-in. ² /ft)	$G_{\text{eff},f,0}$ (10^6 lbf/ft)	$G_{\text{eff},f,90}$ (10^6 lbf/ft)
Topping	1.5	18	17.1*	17.1*	36.5**	36.5**
6 7/8-in. 5-ply V2 CLT	6.875	16	363	81	0.91	1.0
Total	8.375	34	380	98	37.4	37.5

* $E_{c,dynIc}$, ** $G_{c,dynAc}$

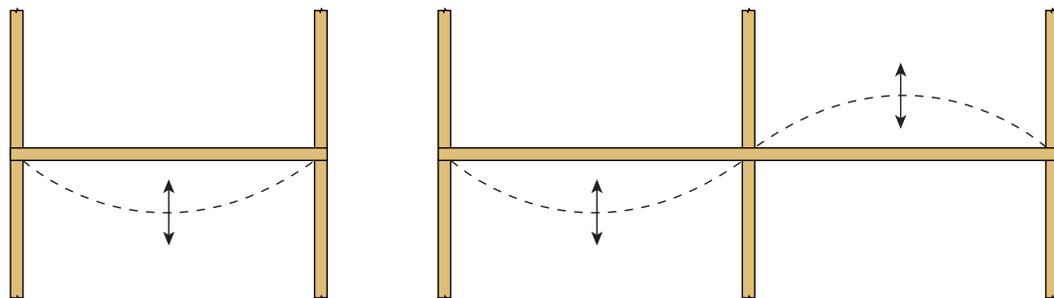


FIGURE 7-3: Simple-span and two-span continuous mode shapes

7.1.2 Damping

The floor has a concrete topping and applied ceiling and the residential units will likely be furnished. Therefore, a moderate damping value of 3% is selected (Section 3.2). Any interior partitions will not reduce the maximum floor panel spans.

7.1.3 Mass

The mass of the system is based on the mass of the CLT floor panels and concrete topping (Table 7-1), plus 12 psf of additional superimposed mass. The additional mass includes 6 psf for the expected portion of the live load per the recommendations in Section 3.1 and 6 psf for mechanical systems and drywall ceiling.

The total expected mass of the floor is 46 psf, which is from the panel (16 psf), topping (18 psf) and superimposed mass (12 psf).

7.1.4 Edge Support

A continuous vertical support is used to model the bearing walls supporting the structure. This treats the walls as being rigid to vertical loads, which neglects the contribution of the walls to the vibration performance of the system. The supports are assumed to freely allow rotation of the floor panels. While in reality the support connection at platform-framed bearing walls may be partially constrained against rotation due to compression from walls above (Section 5.5), it is difficult to quantify the stiffness of this connection. It is therefore conservative to approximate the connection as a pin, which is recommended in the absence of appropriate test data. The perimeter and corridor walls are also modeled as fixed vertical supports.

7.1.5 Walking Frequency

A maximum walking frequency of 1.85 Hz is considered for the analysis. This value conforms with guidance from Section 3.6.1, based on the floor plate size and occupancy type. Because the floor plate is small, relatively enclosed, and low pedestrian traffic is assumed, it is unlikely that walking frequencies will reach higher than 1.85 Hz for a sustained length of time.

7.2 Simplified Methods

Simplified hand calculation methods can provide early insight regarding how the floor system is expected to perform.

7.2.1 Frequency Check

The natural frequency of a 1-ft-wide strip of floor spanning 15 ft is calculated as follows:

$$f_n = 0.18 \sqrt{\frac{g}{\Delta}} = 0.18 \sqrt{\frac{386.4 \text{ in./sec}^2}{0.138 \text{ in.}}} = 9.5 \text{ Hz}$$

Where the deflection due to the estimated mass is:

$$w = 46 \text{ psf} \left(\frac{1 \text{ ft}}{12 \text{ in.}} \right) * 1 \text{ ft width} = 3.83 \frac{\text{lbf}}{\text{in.}}$$

$$L = 15 \text{ ft} \left(\frac{12 \text{ in.}}{1 \text{ ft}} \right) = 180 \text{ in.}$$

$$EI = EI_{\text{eff},f,0} = 380 * 10^6 \text{ lbf-in.}^2/\text{ft}$$

$$\Delta_M = \frac{5wL^4}{384EI} = \frac{5(3.83 \text{ lbf/in.})(180 \text{ in.})^4}{384(380 * 10^6 \text{ lbf-in.}^2/\text{ft})} = 0.138 \text{ in.} \left[\frac{L}{1304} \right]$$

Resonant response (Section 4.3.1) is of concern when $f_n < 4f_w + 2 \text{ Hz} = 4 * 1.85 \text{ Hz} + 2 \text{ Hz} = 9.4 \text{ Hz}$.

Therefore, the estimated frequency of 9.5 Hz suggests that the floor will provide satisfactory results against resonant response criteria. Transient response (Section 4.3.2) is of concern when $f_n > 4.0 f_w = 7.4 \text{ Hz}$; therefore, further investigation of the transient response is warranted.

7.2.2 Live Load Deflection

The design live load deflection of a 1-ft-wide strip of floor spanning 15 ft is calculated as follows:

$$\Delta_L = \frac{5w_L L^4}{384EI} = \frac{5(3.33 \text{ lbf/in.})(180 \text{ in.})^4}{384(363 * 10^6 \text{ lbf-in.}^2/\text{ft})} = 0.125 \text{ in.} \left[\frac{L}{1436} \right]$$

Where:

$$w_L = 40 \text{ psf} * \frac{1 \text{ ft}}{12 \text{ in.}} * 1 \text{ ft width} = 3.33 \frac{\text{lbf}}{\text{in.}}$$

$$EI \text{ of the panel alone} = 363 * 10^6 \text{ lbf-in.}^2/\text{ft}$$

The check of live load deflections reveals that the selected panel is well above code limits for design live load deflection consistent with expectations for a high-performing floor.

7.3 CLT Handbook Method

For an evaluation of the floor in this example, the CLT Handbook method can be applied. From Table 4-4, the maximum vibration span recommendation of 6 7/8-in. 5-ply V2 in a single-span condition on bearing walls is 17.2 ft. Therefore, in a single-span condition, the selected CLT panel is within the recommendations of the CLT Handbook method.

Heavy toppings can have an adverse effect on a floor’s vibration performance by reducing its fundamental frequency. The CLT Handbook method allows for the maximum vibration span to be used without modification for toppings that weigh less than twice the panel weight (i.e., topping weight $\leq 2 * \text{panel weight}$). If this condition is not met, a reduction of up to 10% of the maximum vibration span is recommended (Section 4.2.2).

7.4 Dynamic Calculations and Post Processing

While the floor complies with the basic assumptions of the simplified method presented in the CLT Handbook given the modest weight of concrete topping and bearing wall supports, a sample calculation is provided to demonstrate the modal response analysis methods. The sample calculation is performed for the center point of the model and demonstrates the implementation of the resonant and transient response calculations per Sections 4.3.1 and 4.3.2.

7.4.1 RFEM Model Overview

The model of the floor system (Figure 7-4) is created with three surface elements representing 10-ft-wide CLT floor panels. The panels are supported with a line hinge support at the CLT bearing wall locations, including exterior and corridor wall locations. Because a concrete topping is provided, the RF-laminate module is not used; instead, each surface element is used with orthotropic material properties defined in Table 7-1 above. The masses for the CLT floor, concrete topping, live load, and mechanical systems (46 psf total) are combined into a single area load and applied to the shell elements. The model input parameters and resulting orthotropic properties are shown in Figure 7-5.

Each panel is connected to adjacent panels with connectors, and the concrete topping is the primary element transferring force between panels; to model this behavior, two small surface elements were added between each slab with only the material properties of the concrete layer. The CLT panels were modeled in the single-span and double-span configurations.

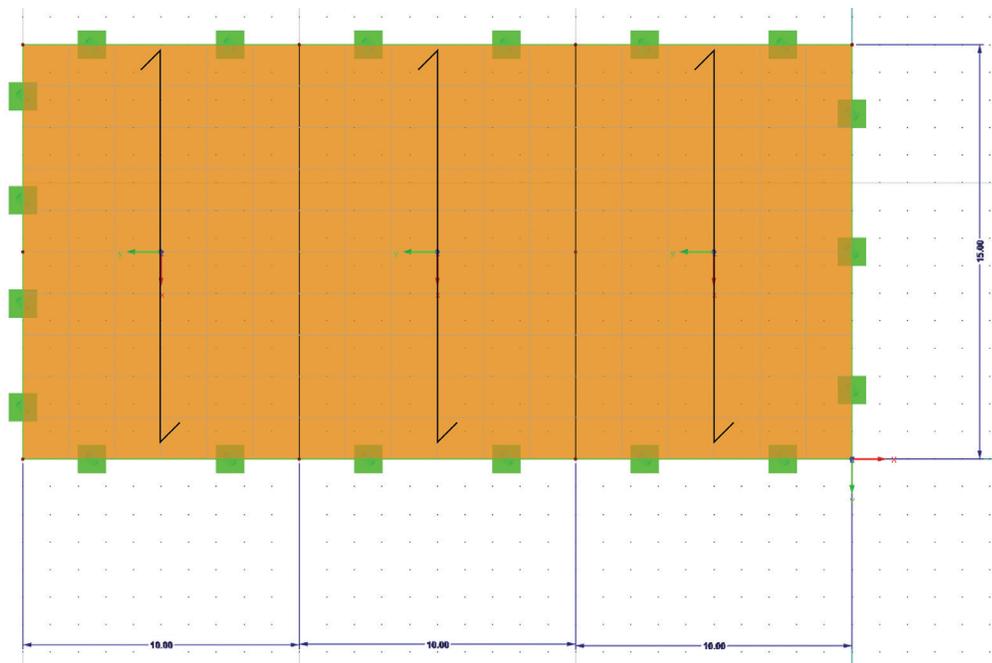


FIGURE 7-4: Single-span FEA model

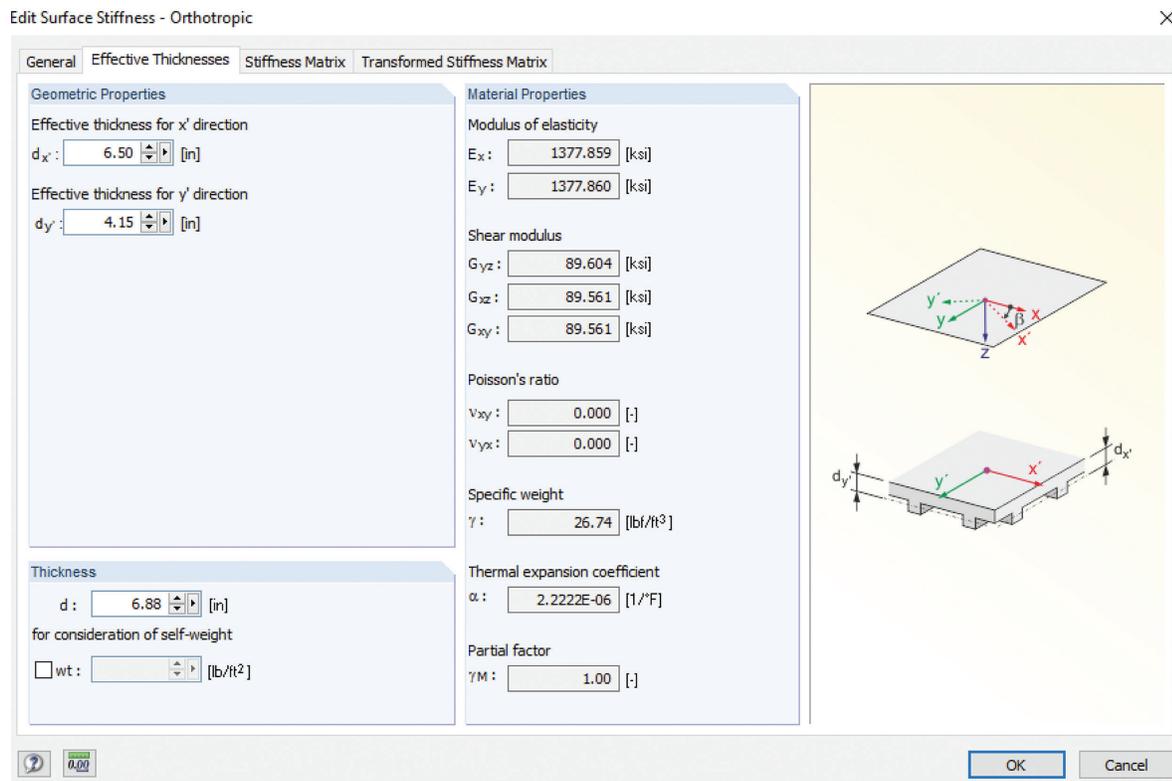
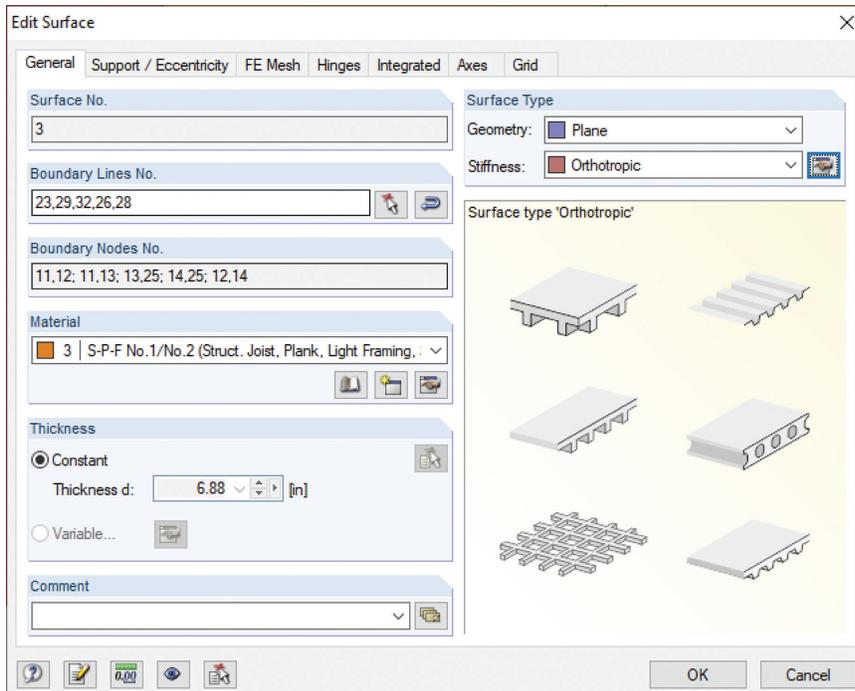


FIGURE 7-5: Orthotropic surface properties

The effective thickness of the orthotropic panel in each direction is determined by summing the panel and topping stiffnesses to obtain a combined, non-composite bending stiffness from Table 7-1, then dividing by E of the selected material, and solving for d_x or d_y for the direction under consideration from the effective I. A sample calculation for d_x is provided below for reference.

$$EI_{\text{eff}_x} = EI_{\text{eff}_0\text{-CLT}} + EI_{\text{eff}_0\text{-conc}} = 380 * 10^6 \text{ lbf-in.}^2/\text{ft}$$

$$E_{\text{ref}} = 1.378 * 10^6 \text{ psi, which is the E of the reference material (SPF) selected in the model}$$

$$I_{\text{eff}_x} = \frac{EI_{\text{eff}_x}}{E_{\text{ref}}} = 275.5 \text{ in.}^4/\text{ft}$$

$$d_x = \sqrt[3]{I_{\text{eff}_x} * \frac{12}{12 \text{ in./ft}}} = 6.50 \text{ in.}$$

The RFEM model was used to calculate the first five modes of vibration for the single-span and double-span configurations (Table 7-2). For both the single-span and double-span models, a fundamental frequency of 9.84 Hz is calculated. The dynamic analysis parameters used in the single-span RFEM model are presented in Figure 7-6. The first three mode shapes for the single-span model are presented in Figure 7-7.

The natural frequency of the floor is in the transitional range between resonant and transient-governed response: $f_n \sim 4f_w$; therefore, it is prudent to perform transient and resonant response analyses. Calculations for these two methods are presented in Section 7.4.

TABLE 7-2: Floor model modal results

	Mode	1	2	3	4	5
Single-span	Modal mass (lbf-s ² /in.)	12.4	12.2	12.8	11.9	9.1
	Frequency (Hz)	9.84	11.17	15.55	21.59	37.80
	Modal displacement*	-1	0	1	0	-0.89
Double-span	Modal mass (lbf-s ² /in.)	24.8	24.4	22.1	25.6	21.7
	Frequency (Hz)	9.84	11.17	15.00	15.55	15.94
	Modal displacement*	-1	0	-0.96	-1.0	0

*Normalized displacement at mid-point of floor

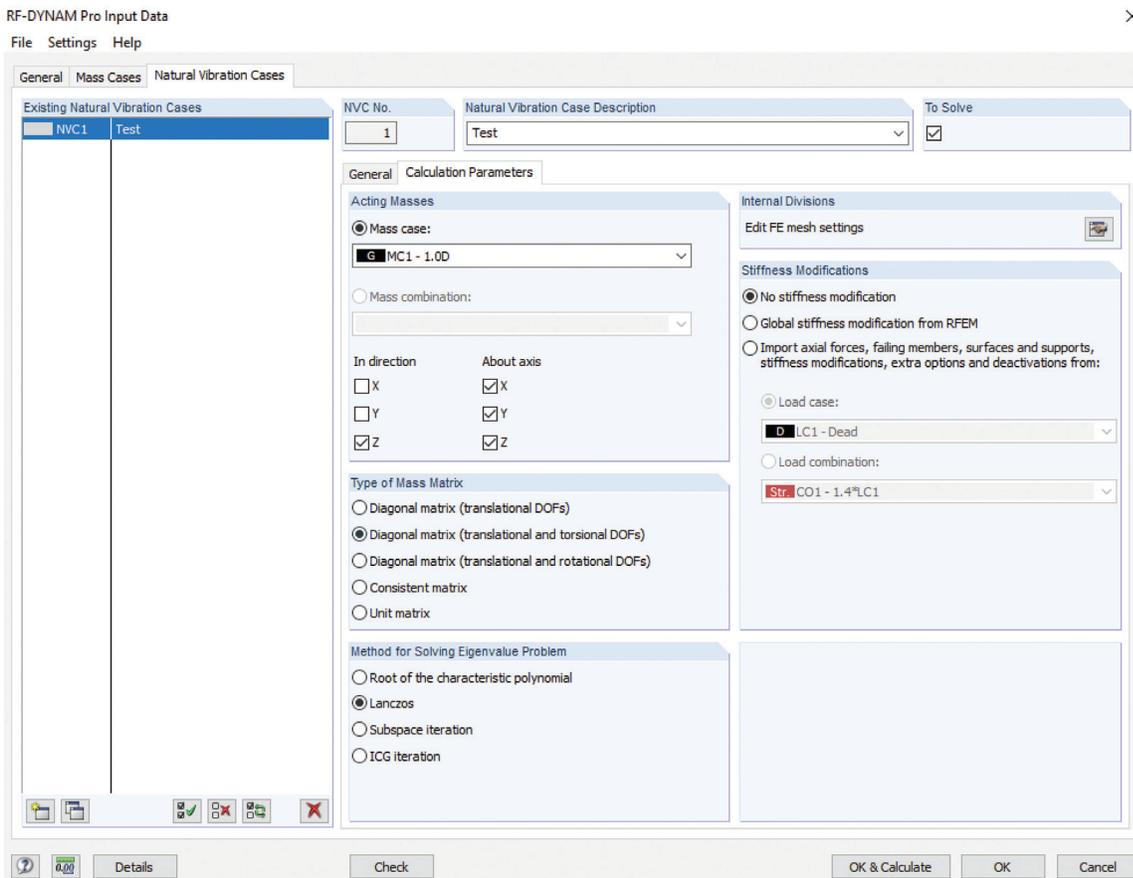
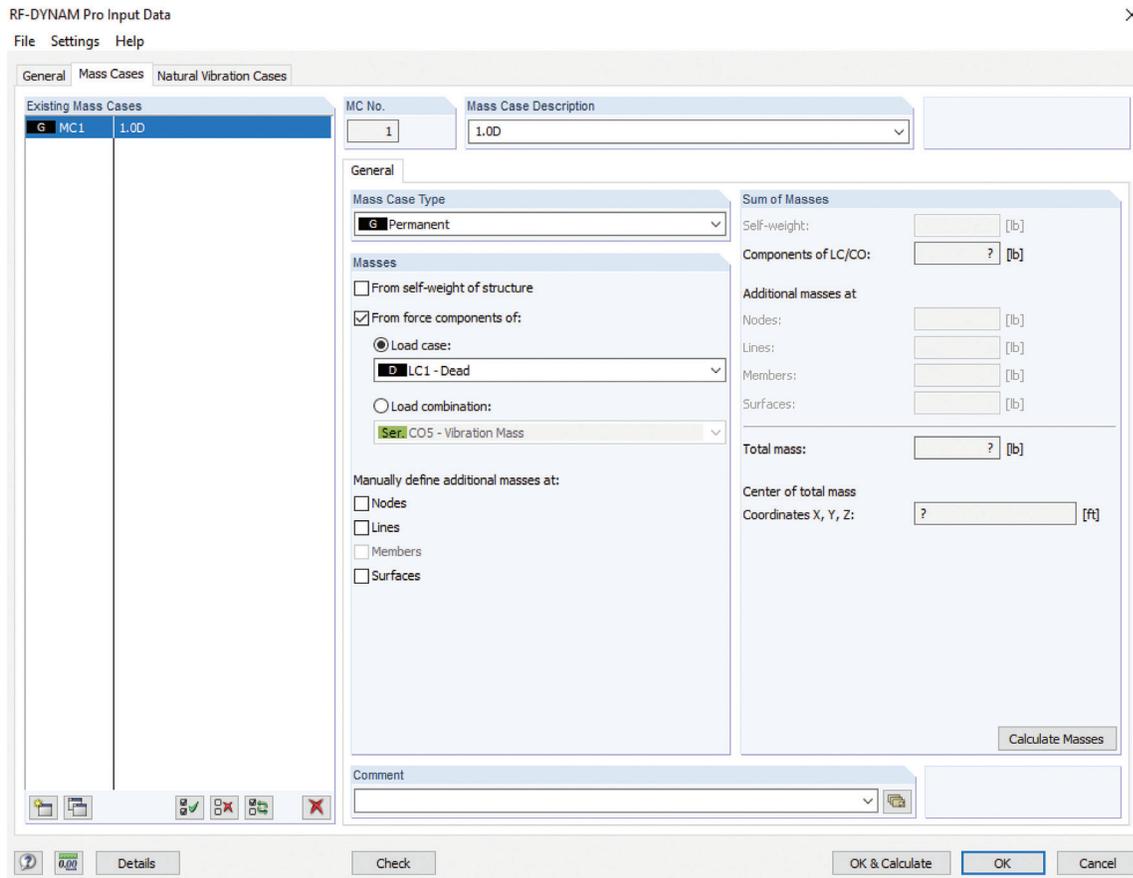


FIGURE 7-6: RF-DYNAM Pro inputs

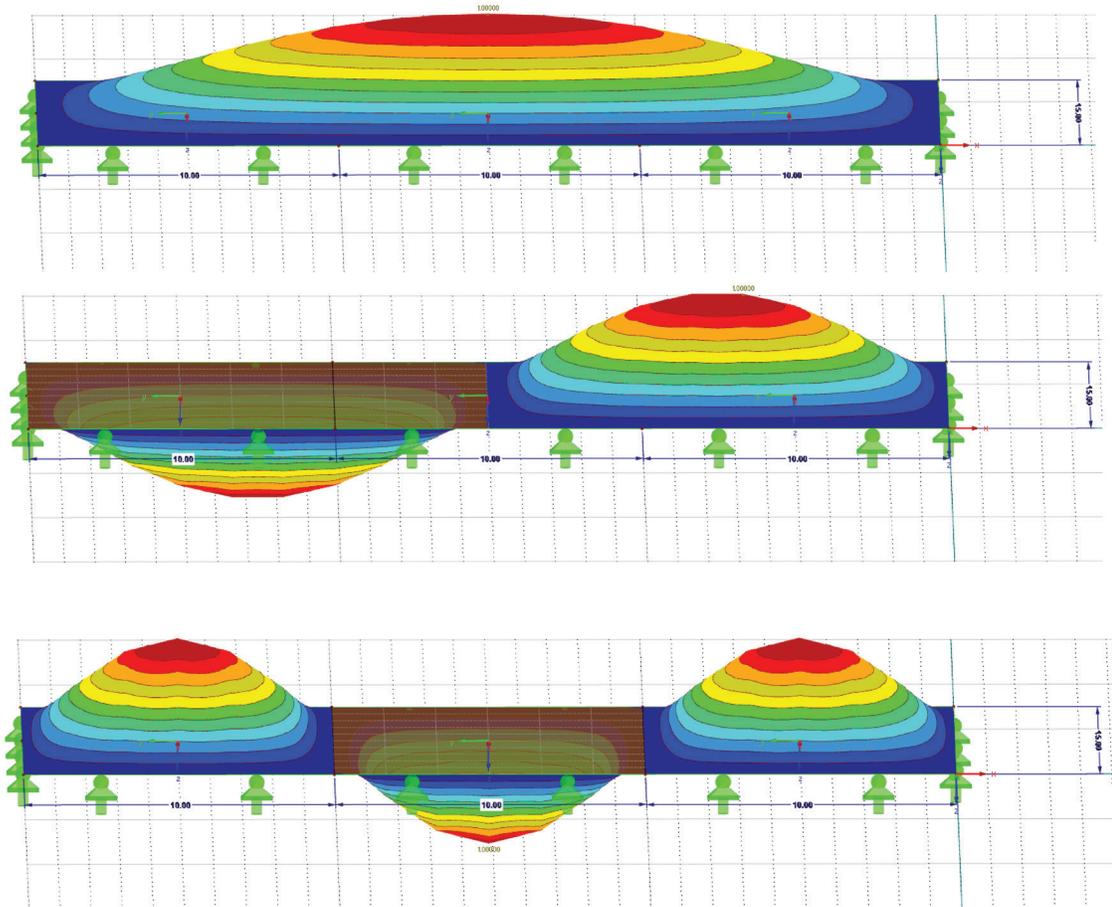


FIGURE 7-7: Single-span first three mode shapes

To simplify modeling, it may be desirable to limit the explicit modeling of panel joints. The impact was explored by deleting panel-to-panel joints in the transverse direction, allowing for a fully rigid joint between panels. In this example, the elimination of transverse panel joints had a minimal effect on the calculated natural frequencies; however, the modal mass increased in the range of 5-10% across several modes. This non-conservative overestimation of modal mass would reduce the predicted accelerations but may not have a significant impact on the overall assessment of the floor’s performance. The elimination of panel joints in the strong axis, however, would effectively turn single-span panels into continuous span panels, and would therefore significantly increase the modal mass estimations. If such joints are present, an assumption of continuous panels is unconservative and could have a significant impact on the assessment of the floor’s performance. The substantial effect of this assumption on the results for this example is shown at the end of this chapter.

7.4.2 Post-Processing

The following sample calculation would typically be completed for many points on the floor. For brevity, this example will include only a single point at midspan using a self-excitation approach, providing a conservative response prediction for the system as discussed in Section 4.3. The excitation parameters as discussed above and derived from Section 3.6 are presented in Table 7-3.

TABLE 7-3: Floor excitation parameters

Damping (ζ_m)	3%
Max walking frequency (f_w)	1.85 Hz
Stride length (l)	2.5 ft
Max walking path length (L)	30 ft
Walker weight (P)	168 lbf

7.4.2.1 Resonant Response Calculation

CCIP-016 suggests that only modes less than 15 Hz need to be considered for this analysis, so modes 1 and 2 are used. The calculation must be run for a number of walking frequencies at this point on the floor. This sample calculation will demonstrate calculations for the fourth harmonic of the highest walking frequency (f_w) and mode 1:

$$h = 4$$

$$m = 1$$

$$f_1 = 9.84 \text{ Hz}$$

$$N_4 = 0.55 h \frac{L}{l} = 0.55(4) \left(\frac{30 \text{ ft}}{2.5 \text{ ft}} \right) = 26.4$$

$$\rho_{4,1} = 1 - e^{-2\pi\zeta_1 N_4} = 1 - e^{-2\pi(0.03)(26.4)} = 0.993$$

The forcing function for resonant footfall (Table 4-3) determines the harmonic coefficient (α_4) for the excitation force.

$$f_h = 4 * f_w = 4 * 1.85 \text{ Hz} = 7.4 \text{ Hz}$$

$$\alpha_4 = 0.013 + 0.0065f_h = 0.013 + 0.0065 * 7.4 = 0.061$$

$$F_4 = \alpha_4 * P = 0.061 * 168 \text{ lbf} = 10.25 \text{ lbf}$$

To determine the real and imaginary components of the acceleration, the calculation is provided for mode 1, with the results for each mode provided in Table 7-4.

$$\mu_{r,1} = \mu_{e,1} = -1$$

$$A_1 = 1 - \left(\frac{f_h}{f_1} \right)^2 = 1 - \left(\frac{7.4 \text{ Hz}}{9.84 \text{ Hz}} \right)^2 = 0.434$$

$$B_1 = 2\zeta_1 \frac{f_h}{f_1} = 2(0.03) \left(\frac{7.4 \text{ Hz}}{9.84 \text{ Hz}} \right) = 0.045$$

$$a_{\text{real},4,1} = \left(\frac{f_h}{f_1} \right)^2 \frac{F_4 \mu_{r,1} \mu_{e,1} \rho_{4,1}}{\hat{m}_1} \frac{A_1}{(A_1^2 + B_1^2)} = \left(\frac{7.4 \text{ Hz}}{9.84 \text{ Hz}} \right)^2 \frac{10.25 \text{ lbf}(-1)(-1)0.993}{12.4 \text{ lbf} - \text{s}^2/\text{in}} \frac{0.434}{(0.434^2 + 0.045^2)} = 1.057 \text{ in./s}^2$$

$$a_{\text{imag},4,1} = \left(\frac{f_h}{f_1} \right)^2 \frac{F_4 \mu_{r,1} \mu_{e,1} \rho_{4,1}}{\hat{m}_1} \frac{B_1}{(A_1^2 + B_1^2)} = \left(\frac{7.4 \text{ Hz}}{9.84 \text{ Hz}} \right)^2 \frac{10.25 \text{ lbf}(-1)(-1)0.993}{12.4 \text{ lbf} - \text{s}^2/\text{in.}} \frac{0.045}{(0.434^2 + 0.045^2)} = 0.109 \text{ in./s}^2$$

$$a_{\text{real},4} = \sum_{m=1}^2 a_{\text{real},4,m} = 1.057 \text{ in./s}^2$$

$$a_{\text{imag},4} = \sum_{m=1}^2 a_{\text{imag},4,m} = 0.109 \text{ in./s}^2$$

TABLE 7-4: Summary of real and imaginary accelerations for each mode

Mode	$a_{\text{real},4,1}$ (in./s ²)	$a_{\text{imag},4,1}$ (in./s ²)
1	1.057	0.109
2	0.0	0.0
Sum	1.057	0.109

Taking the SRSS of the of the real and imaginary accelerations for each harmonic, we can determine the expected resonant acceleration of the floor system at the harmonic of the walking frequency (i.e., at $h = 4$ in this example).

$$a_4 = \sqrt{a_{\text{real},4}^2 + a_{\text{imag},4}^2} = \sqrt{(1.057 \text{ in./s}^2)^2 + (0.109 \text{ in./s}^2)^2} = 1.063 \text{ in./s}^2$$

The acceleration caused by the fourth harmonic of the considered walking frequency is converted to a frequency-weighted acceleration per Section 4.3.1.

$$f_h = 7.4 \text{ Hz}$$

$$a_{4,\text{fw}} = a_4 \times \left\{ \begin{array}{l} \frac{\sqrt{f_h}}{2} \text{ if } f_h < 4 \text{ Hz} \\ 1 \text{ if } 4 \text{ Hz} \leq f_h \leq 8 \text{ Hz} \\ \frac{8}{f_h} \text{ if } f_h > 8 \text{ Hz} \end{array} \right\} = 1.063 \frac{\text{in.}}{\text{s}^2} * 1 = 1.063 \text{ in./s}^2$$

This calculation is performed for each mode and each harmonic. The values are summarized in Table 7-5.

TABLE 7-5: Accelerations per harmonic

Harmonic	a_h (in./s ²)	$a_{h,\text{fw}}$ (in./s ²)
1	0.130	0.088
2	0.181	0.174
3	0.421	0.421
4	1.063	1.063

$$a_{p,\text{fw}} = \sqrt{\sum_{h=1}^4 (a_{h,\text{fw}})^2} = 1.160 \frac{\text{in.}}{\text{s}^2} = 0.30\% \text{ g}$$

Following the method above, the predicted peak acceleration for the double-span case is 0.15% g. These values compare favorably with the peak acceleration target of 0.5% g for normal occupancies in Section 3.7.

By repeating these calculations for multiple walking frequencies, we can develop a plot (Figure 7-8) to determine the walking frequency for which the largest response occurs. Based on this analysis, the maximum response occurs at the maximum considered walking frequency of 1.85 Hz for both the single- and double-span floors, which makes intuitive sense since it is closest to a harmonic of the fundamental frequency.

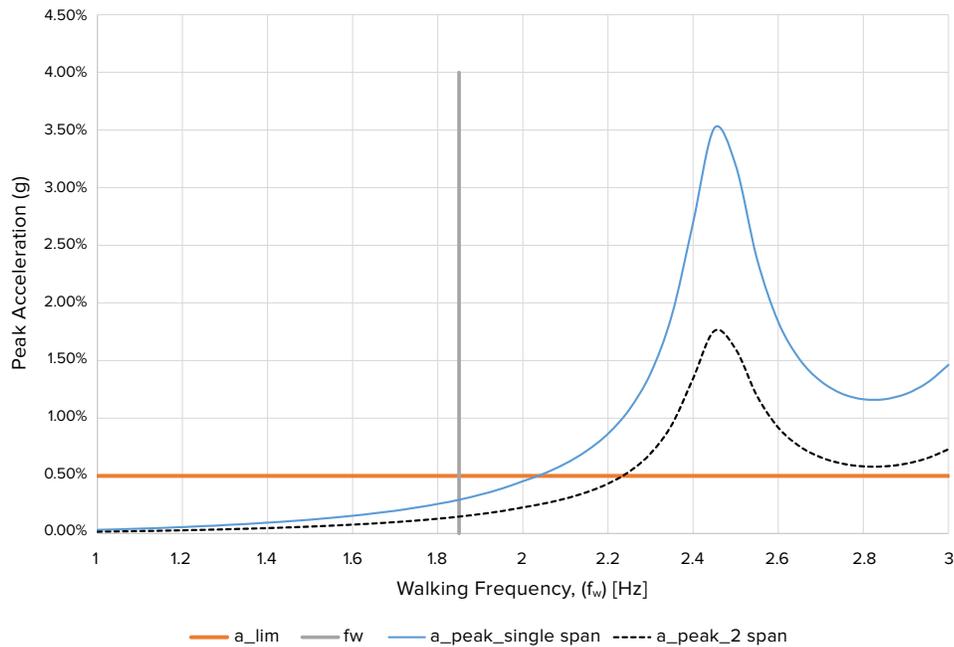


FIGURE 7-8: Peak acceleration for varying walking frequency for self-excitation at the midspan

7.4.2.2 Transient Response Calculation

The floor has a fundamental frequency of 9.84 Hz, so following the recommendations in Section 4.3.2, the calculation of transient response should include contributions from modes up to 19.7 Hz. Considering occupant sensitivity in one-third octave bands, the following shows the calculations for the controlling one-third octave band centered on 10.08 Hz, which contains mode 1 frequency of 9.84 Hz and mode 2 frequency of 11.17 Hz. Only the highest walking frequency is considered for high-frequency floors (Table 7-3). For mode 1, the calculation is as follows:

$$I_{\text{eff},1} = \frac{P}{17.8} \frac{(f_w)^{1.43}}{(f_1)^{1.3}} = \frac{168}{17.8} \frac{(1.85)^{1.43}}{(9.84)^{1.3}} = 1.164 \text{ lbf-s}$$

$$\hat{v}_1 = \mu_{r,1} \mu_{e,1} \frac{I_{\text{eff},1}}{\hat{m}_1} = (-1)(-1) \frac{1.164 \text{ lbf-s}}{12.4 \text{ lbf-s}^2/\text{in.}} * \frac{10^6 \text{ micro-in./s}}{\text{in./s}} = 93,764 \text{ micro-in./s}$$

$$v_1(t) = \hat{v}_1 e^{-2\pi\zeta f_1 t} \sin(2\pi f_1 t) = 93,764 \text{ micro-in./s} * e^{-2\pi(0.03)(9.84\text{Hz})t} \sin(2\pi(9.84 \text{ Hz})t) \text{ micro-in./s}$$

$$v(t) = \sum_{m=1}^N v_m(t)$$

This equation produces a velocity time history based on the initial impulse and must be repeated for all considered modes. The total duration of the time history should equal the time between footsteps, T_w , or $1/f_w$. In this example, Δt is set to T/N where $T = 1/1.85 \text{ Hz} = 0.54 \text{ sec}$, and $N = 200$, which yields satisfactory resolution. This duration should be small enough so as not to alter the result. Repeating the calculation for each mode within the one-third octave band, the sum of the velocities is then determined. To determine the RMS velocity (v_{RMS}), the integral is calculated numerically by summing the results over the duration of the time history. For the single span case:

$$v_{\text{RMS}} = \sqrt{\frac{1}{T_w} \int_0^{T_w} (v(t))^2 dt} \cong \sqrt{\frac{1}{T_w} \sum_{i=0}^N (v(i * \Delta t))^2 \Delta t} = 43,634 \text{ micro-in./s}$$

The RMS velocity for the double-span case is 21,815 micro-in./s. Note that, in accordance with Section 4.3.2, frequency weighting is applicable. For this example, the frequency weighting value is unity since all frequencies exceed 8 Hz. Plots of the transient velocity time history response and v_{RMS} for the single- and double-span models are presented in Figure 7-9.

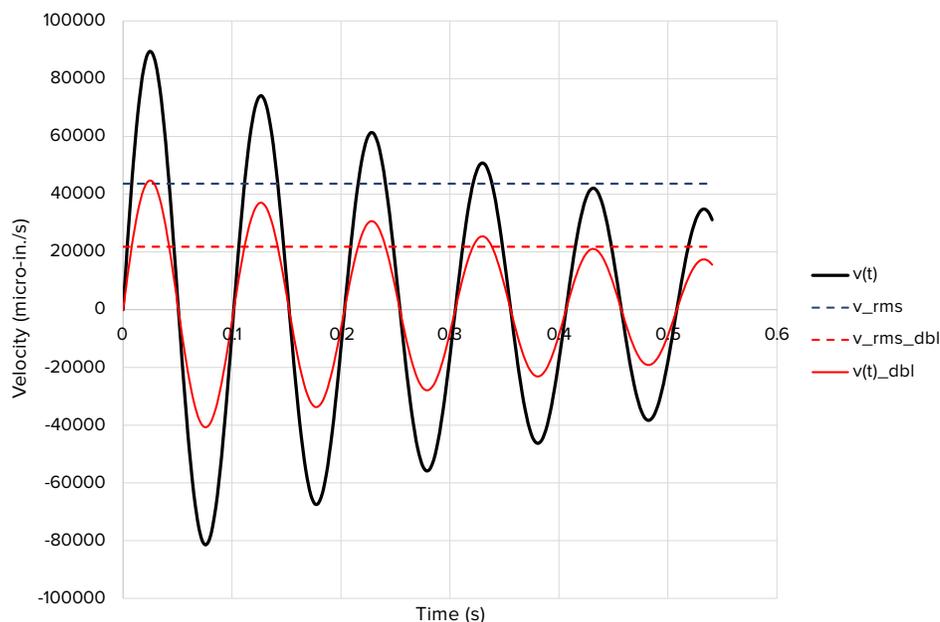


FIGURE 7-9: Velocity time history and V_{RMS} for single-span and double-span transient response

7.5 Results

The 6 7/8-in. V2 panel was calculated as both a simple and continuous panel to demonstrate the impact on the floor's predicted performance and demonstrate the resonant and transient modal response methods. As can be seen in Table 7-6 below, the provision of a continuous span significantly improves the floor's performance due to the additional contributing mass. A 50% reduction in calculated acceleration and RMS velocity response due to continuous spans is consistent with observations in the field and with the expected correlation between participating mass and dynamic response. Should the natural frequency be reduced, or the walking frequency be increased, the predicted accelerations would increase and potentially exceed the performance targets.

The peak acceleration targets for low-frequency floors are discussed in detail in Section 3.7. A target of 0.5% g is generally recommended. Both the single and double-span floors are well below this target and are therefore not susceptible to occupant discomfort due to resonance.

The RMS velocity limits for high-frequency floors are also discussed in detail in Section 3.7. A target of 16,000 to 32,000 micro-in./s for residential occupancies is generally recommended. The simple-span estimate of RMS velocities exceeds the higher end of the range; however, some industry studies of light-frame joisted floors have measured RMS velocities in excess of 40,000 micro-in./s on bare structural floors that were deemed to be satisfactory in practice once construction was complete and the building occupied with typical contents. Furthermore, experience has shown the CLT Handbook method does provide satisfactory performance when applied within the prescribed limitations. The relatively different results between the CLT Handbook method and the transient response analysis highlights the difficulty in applying exact pass/fail criteria to vibration response and demonstrates the role that engineering judgment must play in applying these methods.

TABLE 7-6: Summary of performance

Floor Slab	Span	F_n (Hz)	$a_{p,fw}$ (%g)	a_p Target (%g)	v_{RMS} (mips)	v_{RMS} Target (mips)	CLT Handbook Classification
6 7/8-in. V2	Single-span	9.84	0.30%	0.50%	43,600	32,000	Pass
	Double-span continuous	9.84	0.15%		21,800		Pass

8

Example 2: Open Floor Plan Example Using RFEM

The structural system considered in this example consists of a glulam post-and-beam frame supporting structurally finger-jointed NLT or DLT panels used in an office occupancy.

The beams are 25-ft, single-span, Douglas-fir glulam. The panels are SPF 2x10s with continuous, finger-jointed laminations, spanning two 20-ft bays. Figure 8-1 shows a model of the 2x3 grid of structural bays considered in the vibration analysis.

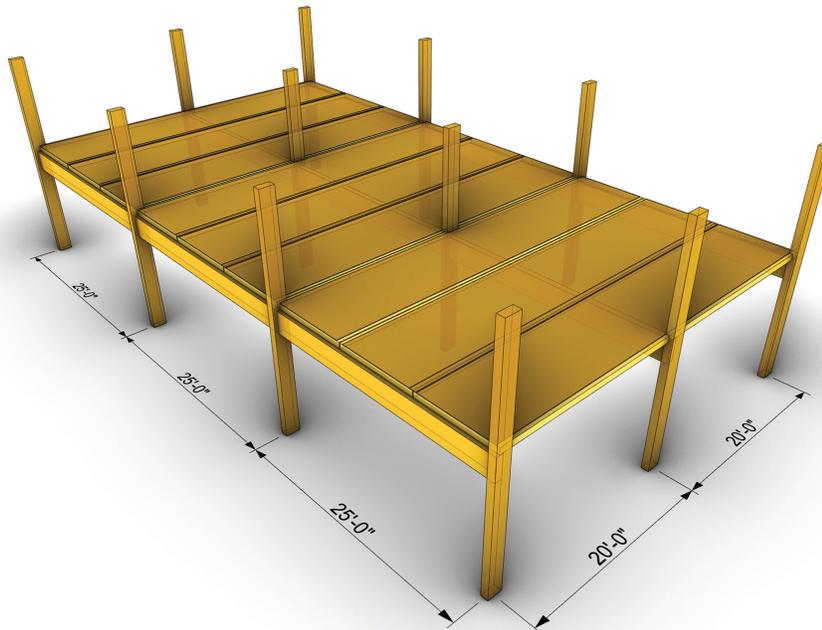


FIGURE 8-1: Structural system considered in this example

The floor build-up consists of the NLT panels topped by 1/2-in. Douglas-fir plywood sheathing, a 3/4-in. acoustic mat and 3-in. of normal weight concrete topping, as shown in Figure 8-2.

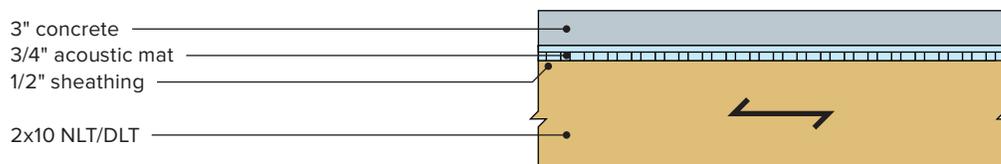


FIGURE 8-2: Floor build-up

This floor system is evaluated using the modal response analysis methods following CCIP-016 as described in Section 4.3. To perform the modal analysis required for these vibration analyses, dynamic results from a finite element model created in RFEM are post-processed using Excel.

8.1 System Properties

The accuracy of modal analysis is highly reliant on correct FE model definition, including material properties, damping, fixity considerations, and contributing superimposed loads on the system.

The following properties have been assumed for this example.

8.1.1 Material Properties

The stiffness of the overall structural floor system is influenced by the stiffness of the beams, panels, and concrete topping. The material properties used in this example are tabulated below.

TABLE 8-1: Material properties

Material	Grade	Size/Thickness (in.)	Specific Gravity G	Modulus of Elasticity E (10 ⁶ psi)	Shear Modulus G _t or G _c (10 ⁶ psi)
Glulam	24F-V4	12 3/8 x 31 1/2	0.48	1.8	0.60
NLT	SPF No. 1/No. 2	9 1/4	0.42	1.4	0.88
Concrete topping	4,000 psi	3	2.4	5.5*	2.25

*Dynamic modulus = 1.35 x E_c per Section 3.3.5

The overall stiffness of the floor is a combination of the panel stiffness and the concrete stiffness. The NLT panels span in one direction, so they do not contribute any transverse stiffness. The independent and combined system properties for the floor assembly are shown in Table 8-2.

TABLE 8-2: System properties

Description	Stiffness, EI _{eff,0} (10 ⁶ lbf-in. ² /ft)	Transverse Stiffness, EI _{eff,90} (10 ⁶ lbf-in. ² /ft)	Shear Stiffness, GA _{eff,0} (10 ⁶ lbf/ft)	Transverse Shear Stiffness, GA _{eff,90} (10 ⁶ lbf/ft)
NLT*	1,108	0	98	0
Concrete topping	140	140	81	81
NLT* + concrete	1,248	140	179	81

*If butt-jointed laminations are used instead of structurally finger-jointed boards, the panel stiffness should be reduced by ~20-30%, as per Table 4.1 of the U.S. NLT Guide (BSLC, 2017)

No composite action between the panel and concrete is considered, as the acoustic mat decouples the panel from the topping. If the concrete is cast directly onto the panel, or there is a positive connection between the concrete and panel, composite action can be considered. Refer to Section 3.4 for additional guidance on composite behavior. Nominal reinforcement of the topping for shrinkage and crack control is assumed. No composite action between the NLT panels and the glulam beams has been assumed.

8.1.2 Loading/Mass

The mass of the system is based on the self-weight of the mass timber panel and concrete topping, a nominal superimposed dead load for electrical/mechanical systems, and an approximate expected live load as per Table 3-1. The estimated mass is shown in Table 8-3.

TABLE 8-3: Estimated mass

Element	Assumed Mass
NLT + sheathing + acoustic mat	22 psf
Concrete topping	38 psf
Mechanical/electrical	4 psf
Expected live load	6 psf
TOTAL	70 psf

8.1.3 Deflection Calculation

Prior to evaluating the vibration performance of the floor plate, it is useful to check the panel deflections. For a two-span panel with the given loading, the deflections are as follows:

Design live load deflection (consider 1-ft unit width):

$$\Delta_L = \frac{w_L L^4}{185 EI} \text{ for uniform load on continuous beam with two equal spans}$$

$$w_L = 65 \text{ psf (1 ft width) (1 ft/12 in.)} = 5.41 \text{ lbs/in.}$$

$$L = 20 \text{ ft} = 240 \text{ in.}$$

$$E = 1400 \text{ ksi}$$

$$I = 791 \text{ in.}^4$$

$$\Delta_L = 0.088 \text{ in.} = L/2740$$

Total (instantaneous) load deflection (consider 1-ft unit width):

$$\Delta_{\text{total}} = \frac{w_{\text{total}} L^4}{185 EI}$$

$$w_{\text{total}} = 129 \text{ plf} = 10.75 \text{ lbs/in.}$$

$$\Delta_{\text{total}} = 0.17 \text{ in.} = L/1380$$

Therefore, the deflections of the panels are well within code limits. Long-term deflection calculations will need to consider the impact of creep, which amplifies the instantaneous deflection.

8.1.4 Frequency Calculation

A useful preliminary check of any structural system is an estimation of its natural frequency, determined using the expected mass as the weight and stiffness.

Deflection of a simply supported span of the panel, under expected loads (Section 4.2):

$$\Delta_{\text{exp}} = \frac{5 w_{\text{exp}} L^4}{384 EI}$$

$$w_{\text{exp}} = 70 \text{ plf} = 5.83 \text{ lbs/in.}$$

$$EI = 1248 * 10^6 \text{ lbf-in.}^2 \text{ per 1-ft strip}$$

$$\Delta_{\text{exp}} = 0.202 \text{ in.}$$

Estimate of natural frequency:

$$f_n = 0.18 \sqrt{\frac{g}{\Delta}} = 0.18 \sqrt{\frac{386.4 \text{ in./s}^2}{0.202 \text{ in.}}} = 7.9 \text{ Hz}$$

Note that as the EI of the panel and topping alone is used in this equation, it is predicting the natural frequency of a simply supported, single-span panel. This example uses multi-span panels on beams, providing flexible vertical support at panel ends and rotational fixity from the panels into the beams. The vertical flexibility of the beams will decrease the natural frequency from this estimate; while the rotational restraint of the panels connected to the beams and columns will increase the natural frequency from this estimate.

The natural frequency of a floor system is only an indicator and does not provide an accurate sole measure of a floor system's vibration performance. A more comprehensive dynamic analysis, as discussed in the following sections, will give a more detailed prediction of this floor system's performance.

8.2 Dynamic Analysis

As discussed in Section 4.3, modal response analysis is a rigorous and precise method of evaluating the vibration performance of a floor. It involves creating a finite element model to derive the dynamic properties of a system, and post-processing the results to determine accelerations and velocities caused by typical walking or heel-drop activities across the floor plate. For this example, a finite element model is created in RFEM and the results are post-processed in an Excel spreadsheet. Hand calculations are performed to validate the outputs of the post-processor.

8.2.1 FEA Model Setup

The floor assembly is modeled with a 9 1/4-in. thick orthotropic surface with modified material properties (see Table 8-2) to capture the influence of the concrete topping. For simplicity, the surface is modeled in the same plane as the centerlines of the beams. This eliminates the need to model connections between the surface and the beam and prevents composite action between the panels (about the weak-axis) and the beams inadvertently being generated. Following the guidance provided in Section 5.5, the panels are modeled as rotationally fixed to the beams, the beams are modeled as fixed at the columns, and the mid-points of the columns are pinned above and below the floor. Figure 8-3 includes two screenshots of the RFEM model used for the modal analysis.

Note that the assumption of rotational fixity between the panels and the beams is potentially non-conservative and should be considered by the design team on a project basis.

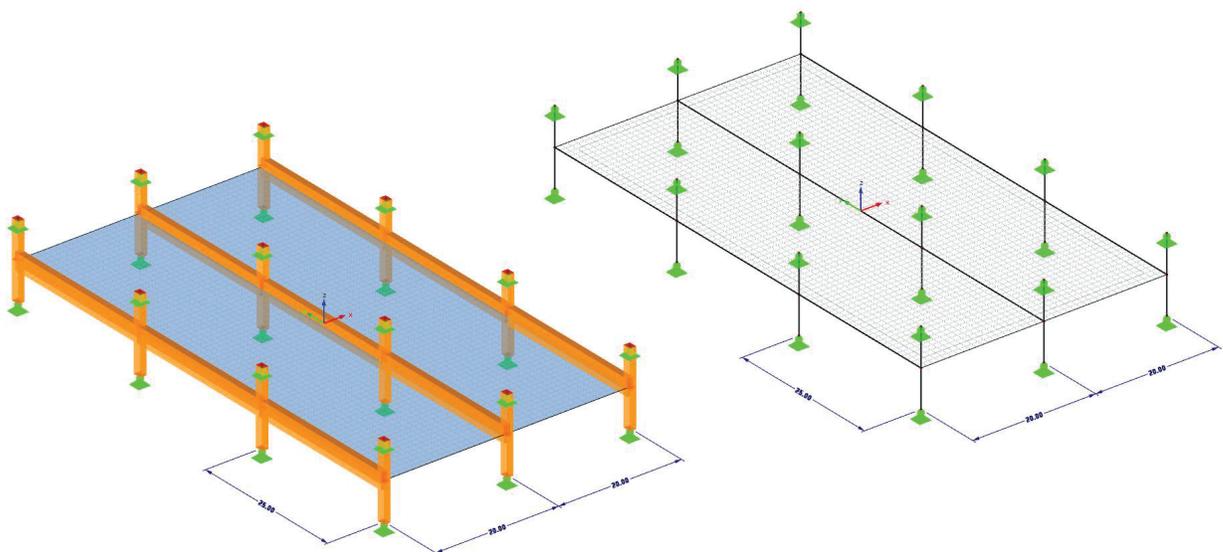


FIGURE 8-3: RFEM model used to analyze the floor assembly*

**Note that the surface is continuous across the middle beam support*

Depending on the detailing of connections between the exterior cladding system and floor plate, it can sometimes be reasonable to add a spring or rigid support along the panel edge. Refer to Chapter 5 for more detailed guidance on creating finite element models for vibration analysis that accurately reflect the structural boundary conditions.

Dynamic analysis involves accurately modeling the stiffness of structural elements, assigning appropriate mass to the system, and using appropriate modal analysis parameters. The mass case includes the self-weight of the model, the weight of the concrete topping and super-imposed dead load, and the live load, of which the latter two are added as surface loads. Vibration analysis is performed in the vertical direction only. For this design example, lateral vibration has negligible contribution to the overall behavior of the floor. The mode shapes are unity normalized. Lastly, 15 modes were calculated, of which eight are considered for resonant response analysis and 13 are considered for transient response analysis, as discussed in the following sections.

8.2.2 FE Model Results

The FE model outputs the natural frequency and modal mass for each mode, as well as the unitless displacements of each node for each mode, representing the dynamic properties of the system. For this example, the frequencies and modal masses are tabulated in Table 8-4.

As shown, the first mode has a frequency of approximately 9 Hz, between $4f_w = 8.8$ Hz and $4f_w + 2$ Hz = 10.8 Hz where the maximum walking frequency is taken as 2.2 Hz. Thus, both resonant and transient responses should be checked, as either may govern depending on floor location being considered. As per Section 4.3.1, only modes less than 15 Hz typically need to be considered for resonant response.

Further, the natural frequency is significantly higher than the natural frequency predicted in Section 8.1.4. This can be attributed to the rotational fixity of the panels to the beams adding stiffness to the system.

The modes shapes of the first two modes are shown in Figure 8-4.

TABLE 8-4: Frequencies and modal masses for each mode

Mode	Frequency, f_m (Hz)	Modal Mass, m_m (lb-f-s ² /in.)
1	9.01	178.1
2	9.56	153.4
3	9.70	122.6
4	9.92	125.3
5	9.93	69.2
6	11.11	92.0
7	13.49	126.0
8	14.24	55.2
9	15.84	52.6
10	16.06	125.2
11	16.31	57.5
12	17.34	49.8
13	17.72	143.1
14	18.29	46.5
15	19.49	38.8

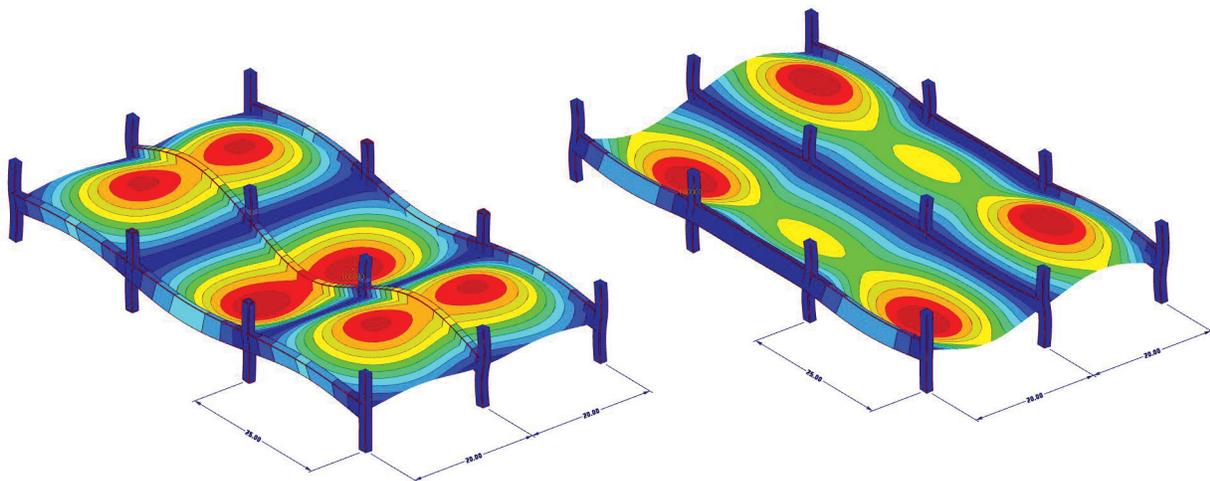


FIGURE 8-4: Mode shapes for mode 1 (left) and mode 2 (right)

8.2.3 Analysis of Results

The FE model results are analyzed as per the methods described in Section 4.3. The weight of the walker is set as 168 lb. For preliminary analysis, walking frequencies between 1.6 Hz and 2.2 Hz are considered—a range appropriate for an open-plan office space.

For simplicity, this example considers only single-point excitation at the worst-case response node. A damping ratio of 2.5% is considered, consistent with recommendations in Table 3-2.

The governing response for this floor occurs at the maximum walking frequency of 2.2 Hz, exciting a node at grid mid-point. It is important to note that excitation at the maximum walking frequency being considered does not always govern resonant response, although it does for transient response. The following hand calculations, per the modal response analysis method of Section 4.3, use this worst-case node as both excitation and

response point. For open-plan offices, excitation and response nodes are usually taken at the same location, as this condition typically governs the response of floor plates. However, in special cases (i.e., if corridors are placed next to meeting rooms), different excitation and response locations should be analyzed, although consideration must be given to the potential of future reconfiguration of open-plan offices. Refer to Section 3.5 for additional guidance.

Analyze Resonant Response

The resonant response, calculated as a frequency-weighted acceleration, for the first harmonic of the walking frequency, based on all the modes less than 15 Hz (eight modes), is calculated as follows:

Frequency of the first harmonic:

$$f_h = hf_w = 1(2.2 \text{ Hz}) = 2.2 \text{ Hz}$$

The forcing function for resonant footfall for the 1st harmonic:

$$\alpha_{h=1} = 0.41(f_h - 0.95) \leq 0.56$$

$$\alpha_{h=1} = 0.41(2.2 \text{ Hz} - 0.95) = 0.51 \leq 0.56$$

$$F_{h=1} = \alpha_1 P = (0.51)168 \text{ lbf} = 86.1 \text{ lbf}$$

Real and imaginary components of acceleration:

$\mu_{r,m}$ and $\mu_{e,m}$ are the modal displacements at the discrete receiver and excitation locations. They are both 1.0 at the peak response point of unity-normalized mode shapes, but not 1.0 for all the modes considered in an analysis.

$\rho_{h,m} = 1$ assumed for all modes and harmonics

$$A_m = 1 - \left(\frac{f_h}{f_m}\right)^2 = 1 - \left(\frac{2.2 \text{ Hz}}{9.01 \text{ Hz}}\right)^2 = 0.940$$

$$B_m = 2\zeta_m \frac{f_h}{f_m} = 2(0.025) \left(\frac{2.2 \text{ Hz}}{9.01 \text{ Hz}}\right) = 0.0122$$

$$a_{\text{real},h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{\hat{m}_m} \frac{A_m}{(A_m^2 + B_m^2)} = \left(\frac{2.2 \text{ Hz}}{9.01 \text{ Hz}}\right)^2 \left(\frac{86.1 \text{ lbf} (1)(1)(1)}{178.1 \text{ lbf s}^2/\text{in.}}\right) \frac{0.940}{(0.940^2 + 0.0122^2)} = 0.0306 \frac{\text{in.}}{\text{s}^2}$$

$$a_{\text{imag},h,m} = \left(\frac{f_h}{f_m}\right)^2 \frac{F_h \mu_{r,m} \mu_{e,m} \rho_{h,m}}{\hat{m}_m} \frac{B_m}{(A_m^2 + B_m^2)} = \left(\frac{2.2 \text{ Hz}}{9.01 \text{ Hz}}\right)^2 \left(\frac{86.1 \text{ lbf} (1)(1)(1)}{178.1 \text{ lbf s}^2/\text{in.}}\right) \frac{0.0122}{(0.940^2 + 0.0122^2)} = 3.97 * 10^{-4} \frac{\text{in.}}{\text{s}^2}$$

Repeating these calculations for modes 2 to 8 results in the accelerations shown in Table 8-5.

TABLE 8-5: Acceleration components for modes 1-8 for $f_h = 1 f_w$

Mode 'm'	$a_{\text{real},h,m}$ in./s ²	$a_{\text{imag},h,m}$ in./s ²
1	3.06E-02	3.97E-04
2	3.12E-02	3.80E-04
3	3.78E-02	4.52E-04
4	3.51E-02	4.10E-04
5	7.49E-03	8.72E-05
6	1.04E-02	1.07E-04
7	3.06E-04	2.56E-06
8	7.07E-05	5.60E-07
Sum	1.53E-01	1.84E-03

Calculate the resonant acceleration for the first harmonic:

$$a_{h=1} = \sqrt{a_{\text{real},h}^2 + a_{\text{imag},h}^2} = \sqrt{\left(0.153 \frac{\text{in.}}{\text{s}^2}\right)^2 + \left(0.00184 \frac{\text{in.}}{\text{s}^2}\right)^2} = 0.153 \frac{\text{in.}}{\text{s}^2}$$

Frequency-weight the acceleration:

$$a_{h,fw} = a_h * \begin{cases} f_h < 4 \text{ hz: } \left(\frac{\sqrt{f_h}}{2}\right) \\ 4 \text{ hz} \leq f_h \leq 8 \text{ hz: } 1 \\ f_h > 8 \text{ hz: } \frac{8}{f_h} \end{cases}$$

For h=1, $f_h = 2.2 \text{ Hz} < 4 \text{ Hz}$ and:

$$a_{1,fw} = a_h * \left(\frac{\sqrt{f_h}}{2}\right) = \left(0.153 \frac{\text{in.}}{\text{s}^2}\right) \left(\frac{\sqrt{2.2}}{2}\right) = 0.11 \frac{\text{in.}}{\text{s}^2}$$

TABLE 8-6: Calculations for the peak acceleration at $f_w = 2.2 \text{ Hz}$

Harmonic No.	1		2		3		4		Normalized displacement at mid-span
Harmonic frequency f_h (Hz)	2.2		4.4		6.6		8.8		
Harmonic force f_h (lbs)	86.1		15.7		12.6		11.8		
Mode m	Real and Imaginary Responses, $a_{\text{real},h,m}$ and $a_{\text{imag},h,m}$ (in./s ²)								μ
	Real	Imaginary	Real	Imaginary	Real	Imaginary	Real	Imaginary	
1	3.06E-02	3.97E-04	2.33E-02	7.48E-04	6.90E-02	5.44E-03	5.46E-01	5.68E-01	-1.000
2	3.12E-02	3.80E-04	2.74E-02	8.01E-04	7.44E-02	4.91E-03	3.91E-01	1.18E-01	-0.997
3	3.78E-02	4.52E-04	3.30E-02	9.41E-04	8.79E-02	5.56E-03	4.16E-01	1.06E-01	-0.997
4	3.51E-02	4.10E-04	3.04E-02	8.39E-04	7.89E-02	4.71E-03	3.30E-01	6.88E-02	0.994
5	7.49E-03	8.72E-05	6.47E-03	1.78E-04	1.68E-02	1.00E-03	6.98E-02	1.44E-02	-0.341
6	1.04E-02	1.07E-04	8.63E-03	2.03E-04	2.03E-02	9.32E-04	5.80E-02	6.17E-03	-0.521
7	3.06E-04	2.56E-06	2.43E-04	4.44E-06	5.16E-04	1.66E-05	1.13E-03	6.42E-05	0.128
8	7.07E-05	5.60E-07	5.58E-05	9.52E-07	1.16E-04	3.42E-06	2.44E-04	1.22E-05	-0.0431
Total	1.53E-01	1.84E-03	1.34E-01	3.85E-03	3.60E-01	2.36E-02	1.91E+00	9.86E-01	
Harmonic response a_h (in./s ²)	1.53E-01		1.34E-01		3.61E-01		2.15E+00		
Freq. weighted harmonic response $a_{h,fw}$ (in./s ²)	1.13E-01		1.34E-01		3.61E-01		1.96E+00		
Peak response $a_{p,fw}$ (in./s ²)	2.00								
Peak response $a_{p,fw}$ (g)	0.52%								

The calculations, repeated for each mode and the remaining three harmonics, are provided in Table 8-6. Finally, the total response $a_{p,fw}$ is calculated as the square-root sum of squares of the frequency-weighted acceleration for each harmonic. If frequency weighting is not applied, a_p is $2.19 \text{ in./s}^2 = 0.57\% \text{ g}$. Note that only the first eight modes are considered in this calculation, as the frequencies of modes 9-15 are greater than 15 hz.

At a walking frequency of 2.2 Hz, the peak acceleration response is 0.52% g, slightly higher than the performance target of 0.5% g for office space described in Section 3.7.2. At the AISC Design Guide 11 “Fast” walking frequency of 2.1 Hz per Table 3-7, the peak acceleration response is 0.34%, significantly under the performance target.

The response of this node under the full range of walking frequencies considered is shown in Figure 8-5.

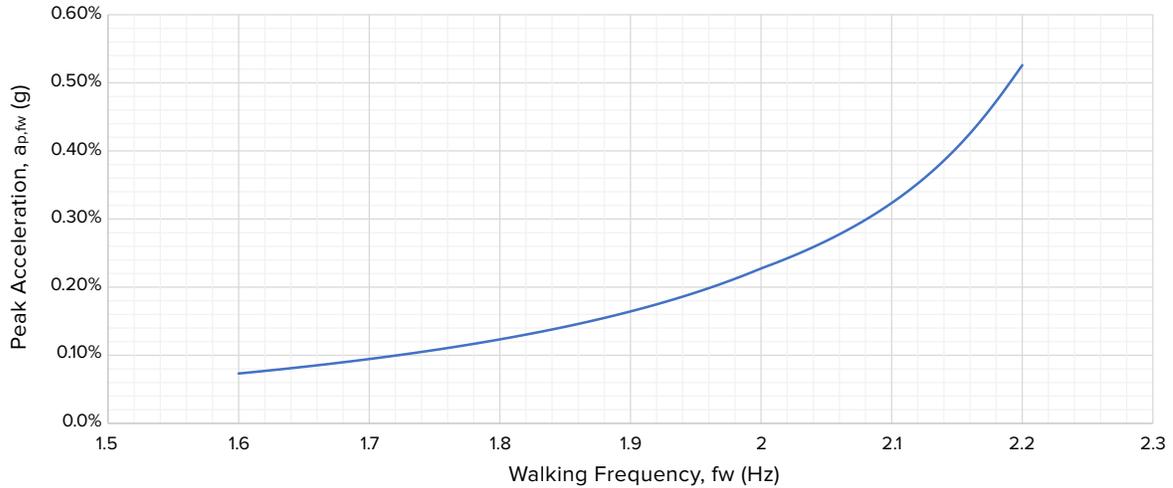


FIGURE 8-5: Peak resonant acceleration vs. walking frequency

Given that the maximum response occurs at a walking pace of 2.2 Hz (132 steps per minute), which exceeds the average walking pace of most individuals, the performance of the floor is deemed acceptable. If higher performance is required, the next step is to evaluate whether composite action is achievable or additional damping from partitions is present. For guidance, refer to Chapter 3.

Analyze Transient Response

The lowest mode has a fundamental frequency of 9.01 Hz, so modes with frequencies less than 18 Hz should be evaluated, per Section 4.3.2. So, for this example, modes 1-13 are considered. The highest walking frequency will govern for transient response calculations. Similar to the calculation for resonant response, the worst-case response node is analyzed, and the excitation node coincides. The calculations for the first mode at time t , are as follows:

$$I_{\text{eff},m} = \frac{P}{17.8} \frac{(f_w)^{1.43}}{(f_m)^{1.3}} = \frac{168 \text{ lbf} (2.2 \text{ Hz})^{1.43}}{17.8 (9.01 \text{ Hz})^{1.3}} = 1.67 \text{ lbf}\cdot\text{s}$$

$$\hat{v}_m = \mu_{r,m} \mu_{e,m} \frac{I_{\text{eff},m}}{\hat{m}_m} = \frac{(1)(1)1.67 \text{ lbf}\cdot\text{s}}{178.1 \text{ lbf}\cdot\text{s}^2/\text{in.}} = 0.0093 \text{ in./s} = 9,300 \text{ micro-in./s}$$

$$v_m(t) = \hat{v}_m e^{-2\pi\zeta f_m t} \sin(2\pi f_m t) = 0.0093 \frac{\text{in.}}{\text{s}} e^{-2\pi(0.025)(9.01 \text{ Hz})t} \sin(2\pi(9.01 \text{ Hz})t)$$

Repeat the calculation for each mode to find the sum of the velocity at time, t :

$$v(t) = v_1(t) + v_2(t) + v_3(t) + \dots + v_{13}(t)$$

Then calculate the root-mean-square velocity:

$$v_{\text{RMS}} = \sqrt{\frac{1}{T_w} \int_0^{T_w} (v(t))^2 dt}$$

$$\text{Where } T_w = \frac{1}{f_w}$$

For this example:

$$v_{\text{RMS}} = 0.0229 \frac{\text{in.}}{\text{s}} = 22,900 \text{ micro-in./s}$$

Lastly, apply frequency weighting to the RMS velocity:

$$\text{If } f_1 > 8 \text{ Hz, } v_{\text{RMS, fw}} = v_{\text{RMS}}$$

$$\text{If } f_1 < 8 \text{ Hz, } v_{\text{RMS, fw}} = v_{\text{RMS}} \frac{f_1}{8}$$

Therefore:

$$v_{\text{RMS, fw}} = v_{\text{RMS}} = 22,900 \text{ micro-in./s}$$

Although the highest walking frequency always governs transient response, it is informative to analyze the transient response across the full range of frequencies. The plot below shows the frequency-weighted response plotted across the full range of walking frequencies.



FIGURE 8-6: Transient velocity response vs. walking frequency

Analyzing the resonant and transient response at every location on the floor plate yields results as shown in Figure 8-7.

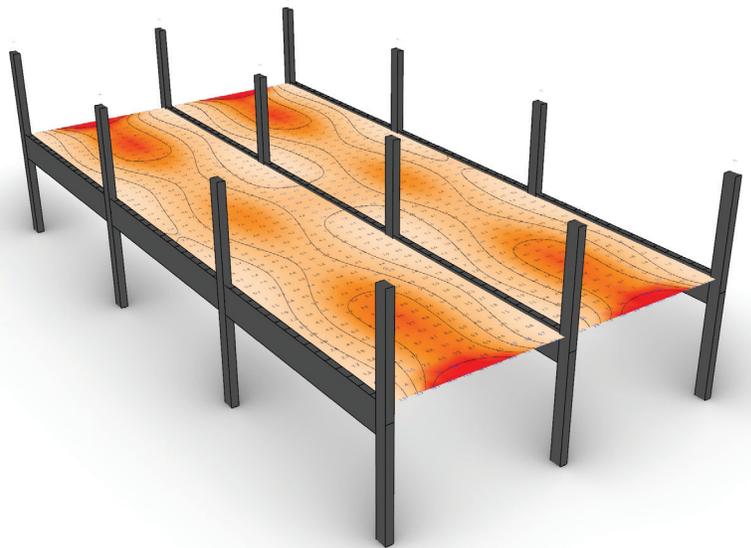


TABLE 8-7: The response factors plotted on the floor plate

8.3 Summary

The modal response analysis techniques presented in Section 4.3 provide a precise way of calculating accelerations on a mass timber floor plate due to footfall excitations. The results are sensitive to both boundary conditions and accuracy of input assumptions, and thus care should be taken in relying on these results for pass/fail decisions. If the floor plate considered in this example is supported by a façade, as shown in Figure 8-8, the response to footfall vibrations is reduced.

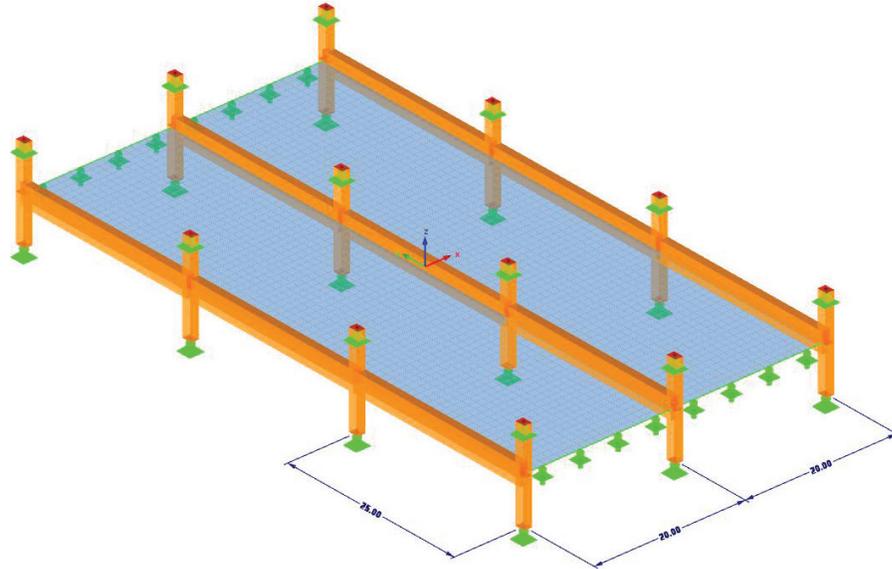


FIGURE 8-8: The floor plate used in this example with vertical spring supports added

The floor plate above yields a maximum frequency-weighted acceleration response of 0.43% g, approximately 17% lower than the same floor plate without façade support.

Designers may also consider using 2x8 finger-jointed NLT panels for this application. Analyzing the same system with 2x8 panels yields a maximum acceleration of 1.14% g, twice as high as the 2x10 system. This value is also over twice the target performance level of 0.5% g and in the field will likely be unacceptable to occupants.

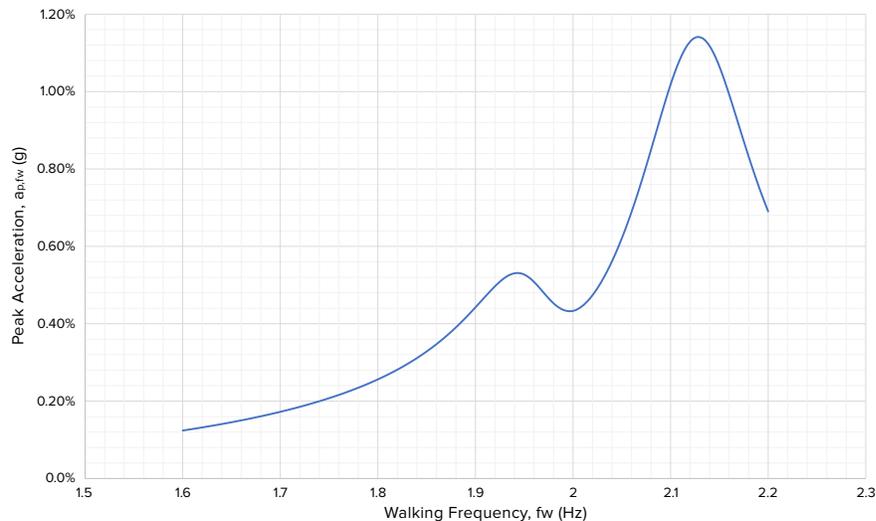


FIGURE 8-9: Peak resonant acceleration vs. walking frequency using 2x8 NLT

To achieve better results, the designer could consider a system capable of achieving composite action, explore whether changing the concrete topping thickness would improve performance, or evaluate whether partitions are present within the interior of the floorplate which could provide additional damping. Refer to Chapter 3 for further guidance.

9

Example 3: High-Performance Floor Using SAP2000

9.1 Introduction

The structural system considered in this example consists of a glulam post-and-beam structure forming a 24x20-ft typical bay supporting CLT floor panels with a concrete topping slab. The floor assembly consists of 4 1/8-in.-thick grade V2 CLT3 panels with (3) 1 3/8-in. laminations topped with 5 in. of normal weight concrete, with composite connection via screws to the CLT panel as shown in Figure 9-1. This floor assembly is significantly heavier than a similar system designed only for strength and deflection because of the need to achieve a very low level of floor vibration. The 5 in. of normal weight concrete has been intentionally chosen to add mass and stiffness to the system; it is similar to the weight necessary for a steel-framed floor with concrete over metal deck to achieve the desired vibration criteria.

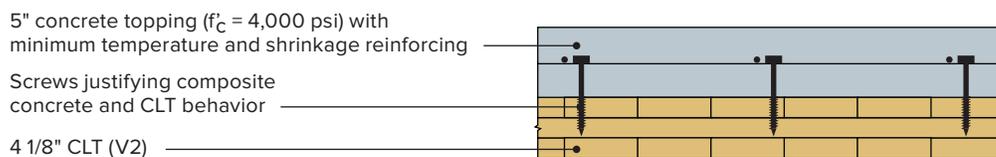


FIGURE 9-1: Floor assembly considered in this example

This example is modeled using SAP2000 to determine modal properties of the floor structure, which are then processed in Microsoft Excel using the modal response analysis methods of Section 4.3. It is intended to show the process for analyzing a high-frequency floor system governed by an impulsive (transient) response, rather than a lower-frequency system with a resonant response-dominated excitation.

The floor is designed to meet a velocity criterion of 4,000 micro-in./s under a 2.0 Hz walking pace. This vibration criterion is appropriate for surgery facilities, operating rooms, and optical microscopes up to 100x magnification (Section 3.7.3).

9.2 Framing Modeling Properties

This example utilizes a 2-bay by 5-bay simple floor plate with geometry as shown in Figures 9-2 and 9-3. Typical glulam framing member properties are shown in Table 9-1. The modeling for this example has been completed utilizing the assumptions listed below. Refer to Chapter 5 for additional information on modeling assumptions and best practices.

- The concrete topping exhibits 50% composite action with the CLT for low-amplitude forces. Connections between CLT and concrete must be specifically detailed to justify composite behavior (Section 3.4.2).
- The CLT panels are fully composite with the glulam framing (Section 3.4.2).
- All beam-to-beam and beam-to-column connections are fully fixed for the purposes of the vibration analysis (Section 5.5).
- The structure exhibits 3.5% of critical damping (Section 3.2).

- Elastic modulus of concrete, E_c , is increased by a factor of 1.35 for dynamic loading (Section 3.3.5).
- Columns are modeled one level above and below using 14-ft floor-to-floor heights (Section 5.3).
- Exterior walls provide vertical spring supports, modeled using springs of stiffness 4 k/in. spaced at 2-ft on center (equivalent to 2 k/in./ft) around the perimeter of the floor plate (Section 5.3).
- Acoustic performance of the floor system has not been considered but would be necessary in a complete design.



FIGURE 9-2 Floor plan showing framing; CLT3 spans left to right

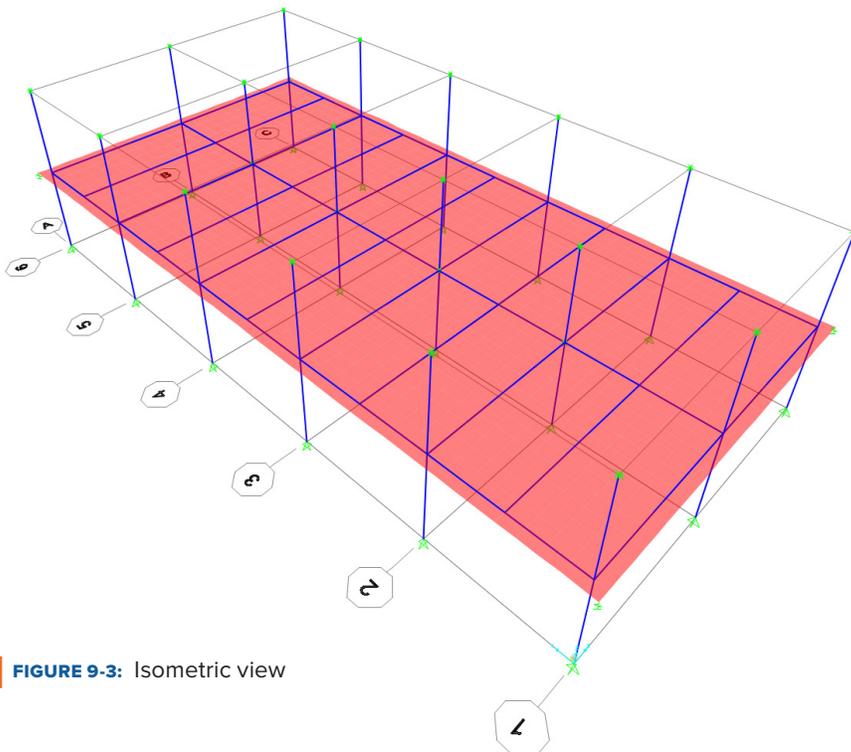


FIGURE 9-3: Isometric view

TABLE 9-1: Typical framing members and properties

Member	Size Required for Strength and Deflection Design Only (100 PSF Live Load)	Size Required for Vibration Design	Grade	Elastic Modulus
Columns	Varies	12 1/4" x 15"	DF L2	1,600 ksi
Beams	10 3/4" x 21"	10 3/4" x 27"	DF 24F-V4	1,800 ksi
Perimeter girders	10 3/4" x 21"	10 3/4" x 30"	DF 24F-V4	1,800 ksi
Interior girders	10 3/4" x 27"	10 3/4" x 30"	DF 24F-V4	1,800 ksi

Glulam beam and girder sizes that have been designed for strength and deflection criteria are included in Table 9-1 for comparison. It is important to note that, even with a heavy 5-in. concrete topping over CLT3 panels and a 100 lb/ft² live load, the glulam framing members require additional depth in order to provide adequate stiffness to the system for the desired vibration performance objective. A 4,000 micro-in./s velocity acceptance criterion is quite stringent regardless of building material, and the cost and assembly depth impacts to the framing system should be recognized.

Table 9-2 shows the properties of the wood laminates used in the CLT3 based on minimum requirements for a V2 CLT layup per PRG 320-2019. Also refer to Table 3-3.

TABLE 9-2: CLT stiffness properties

Lamination	E ₀	E ₉₀ = E ₀ /30	G = E ₀ /16
Major strength direction	1,400 ksi	47 ksi	88 ksi
Minor strength direction	1,200 ksi	40 ksi	75 ksi

9.3 Floor Shell Modeling Properties

To account for the orthotropic nature of the CLT and concrete floor assembly, the floor deck is implicitly modeled as an isotropic material with thickness matching the total CLT plus topping slab thickness. Shell stiffness modifiers f_{11} , f_{22} , f_{12} , m_{11} , m_{22} , m_{12} , ν_{13} and ν_{31} are then used to adjust the program calculated values to an effective stiffness for each applicable loading condition. See Figure 9-4 and note that local axis 1 of the shell elements has been assigned in the model for this example corresponding to the strong direction of the CLT (i.e., perpendicular to the glulam beams) per Figure 9-2. The calculations in the subsequent section show the derivation of each of the shell stiffness modifiers. See Sections 3.3, 3.4, 5.4 and 5.6 for additional background information. The floor shell is modeled with the input properties shown in Figures 9-5 and 9-6. Note that, for simplicity of calculation of the property modifiers, the Poisson's ratio of the shell material is set to zero in SAP2000. The bending stiffness of a plate/shell simplifies when the Poisson's ratio is zero. Glulam beams and girders are modeled explicitly with distinct frame elements. The insertion point for these members is identified as the top-center with the intent that the beam will act fully composite with the floor assembly using rigid offsets. Similarly, the floor shell elements have rigid end offsets applied to set the elevation equal to the center-of-stiffness of the CLT-concrete assembly for appropriate composite interaction with the glulam beams and girders. The shell element offset for this example is modeled as the average neutral axis elevation by averaging \bar{z}_{11} and \bar{z}_{22} calculated in the derivation for f_{11} and f_{22} , respectively, in Section 9.3.1. Application of rigid end offsets in SAP2000 are shown in Figures 9-7 and 9-8.

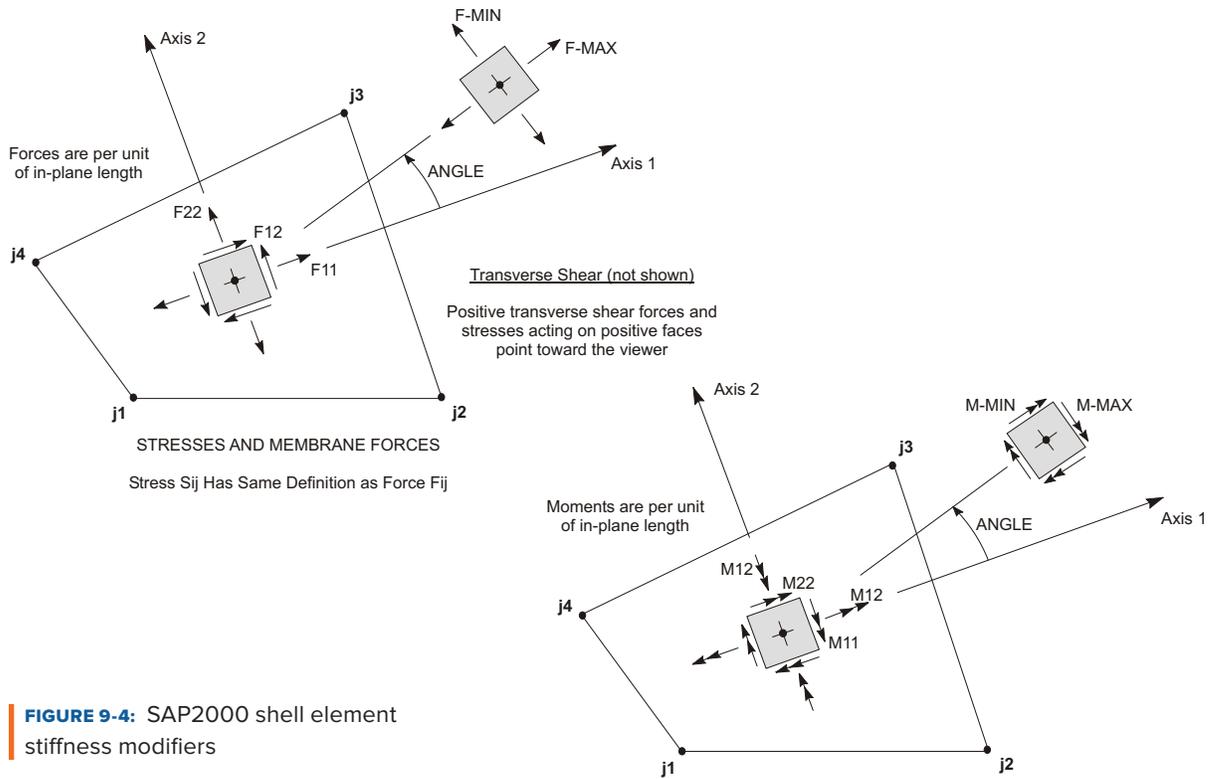


FIGURE 9-4: SAP2000 shell element stiffness modifiers

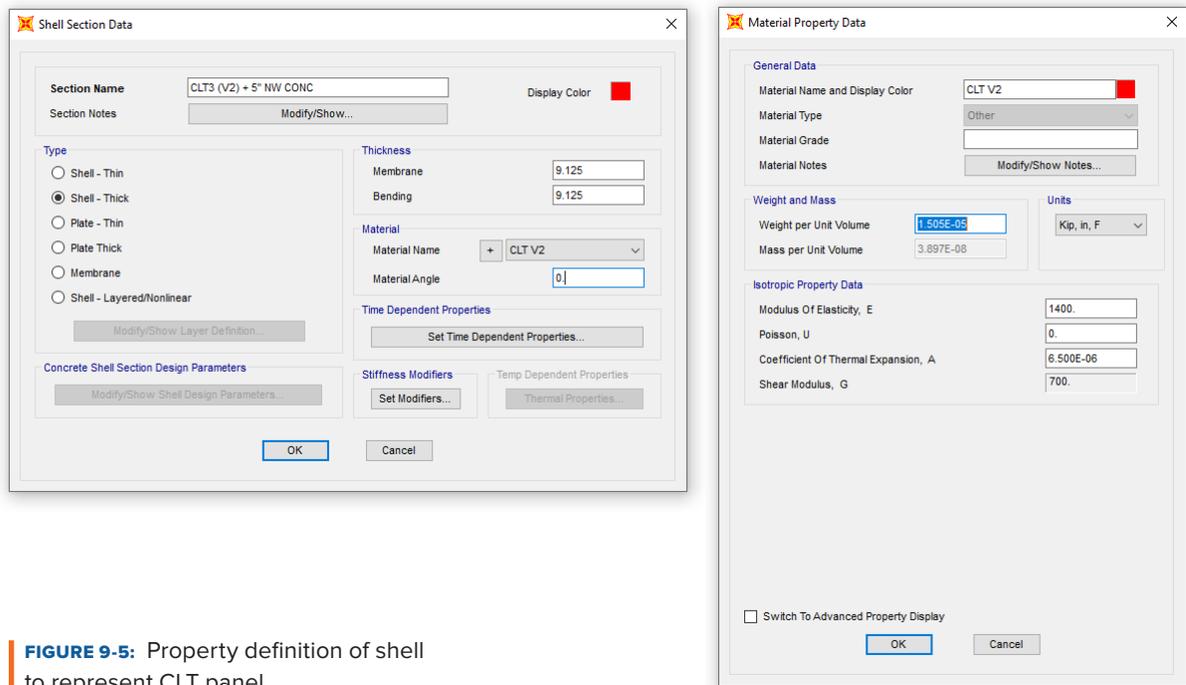


FIGURE 9-5: Property definition of shell to represent CLT panel

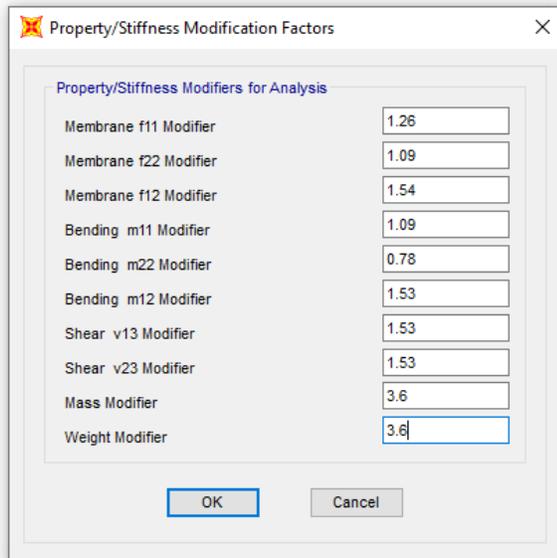


FIGURE 9-6: Shell property/stiffness modification factors

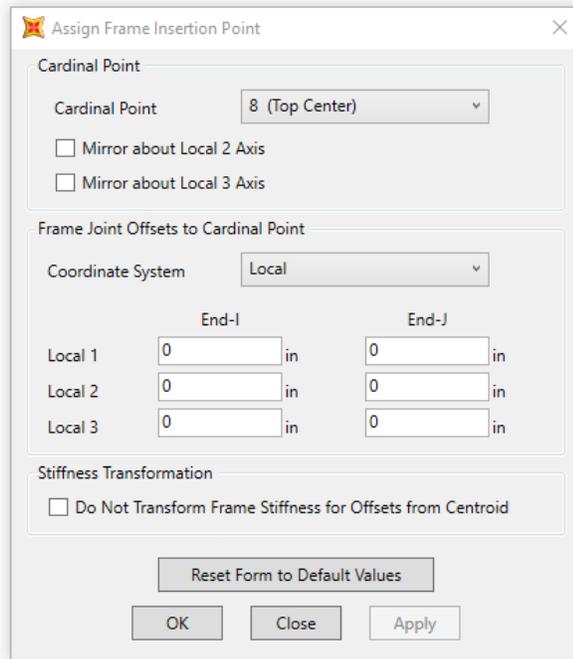


FIGURE 9-7: Beam and girder frame insertion point

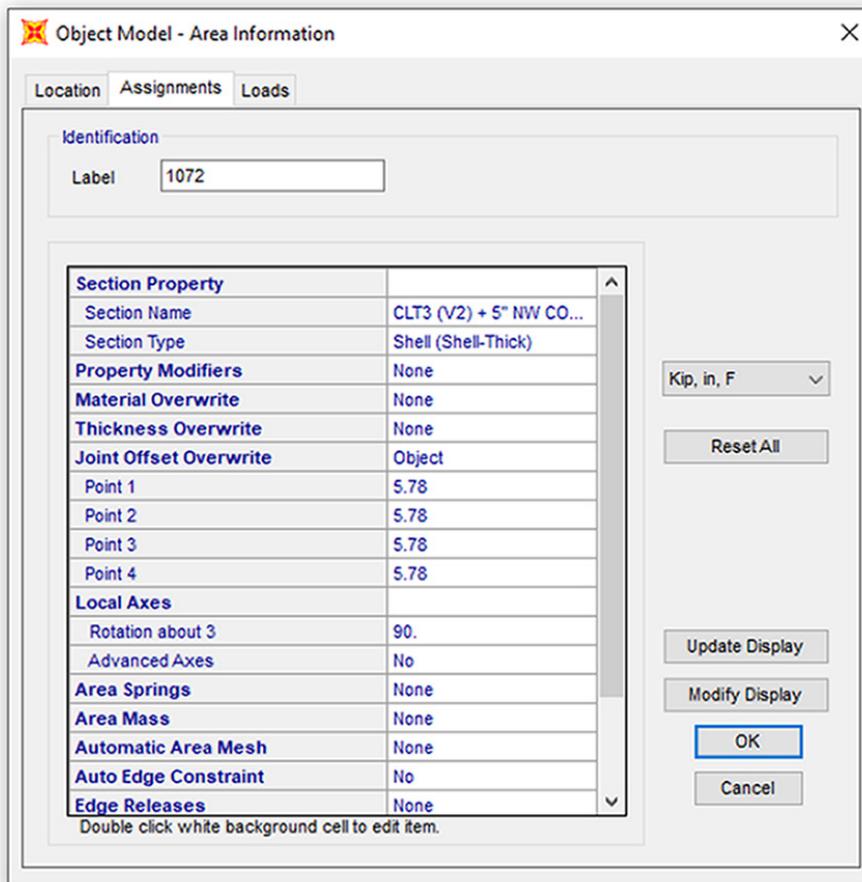


FIGURE 9-8: Slab shell element assignments with joint offset overwrites

9.3.1 Membrane f11 and f22 Modifier Derivation

Modifiers f11 and f22 correspond to in-plane axial stiffness for the shell local 1 and 2 axes, respectively. See Figure 9-4. These modifiers will affect the CLT-concrete assembly bending stiffness when acting composite with the glulam beams and girders. The elastic modulus of concrete, E_c , is increased by a factor of 1.35 per Section 3.3.5 for low-amplitude dynamic loading. This value is then multiplied by the effective composite action factor, γ , to produce $E_{c,eff}$. Note that without applying γ in calculating $E_{c,eff}$, the composite action between the glulam beams and the concrete, by way of the CLT panels, would be over-predicted. Since the CLT and concrete are modeled using rigid end offsets with respect to the glulam beams in this example, not applying γ to $E_{c,eff}$ would be equivalent to assuming 100% composite action of the concrete with the glulam beam.

$$(EA)_{11,SAP} = (EA)_{22,SAP} = (1,400 \text{ ksi})\left(9\frac{1}{8} \text{ in.}\right)(12 \text{ in./ft}) = 153,000 \text{ k/ft}$$

$$(EA)_{C,eff} = 1.35 * 57\sqrt{f'_c} * \gamma * t_c * b = 1.35 * 57\sqrt{4,000 \text{ psi}} * (0.5)(5 \text{ in.})(12 \text{ in./ft}) = 146,000 \text{ k/ft}$$

$$(EA)_{0,CLT} = \sum \left(t_0 E_0 b + \frac{t_{90} E_0 b}{30} \right) = (2)(1.375 \text{ in.})(1,400 \text{ ksi})(12 \text{ in./ft}) + \frac{(1)(1.375 \text{ in.})(1,200 \text{ ksi})(12 \text{ in./ft})}{30} = 46,900 \text{ k/ft}$$

$$(EA)_{11,eff} = (EA)_{C,eff} + (EA)_{0,CLT} = 193,000 \text{ k/ft}$$

$$f11 = \frac{(EA)_{11,eff}}{(EA)_{11,SAP}} = 1.26$$

$$\bar{z}_{f11} = \frac{(EA)_{C,eff} * \bar{z}_C + (EA)_{0,CLT} * \bar{z}_{CLT}}{(EA)_{C,eff} + (EA)_{0,CLT}} = 5.52 \text{ in.}$$

$$(EA)_{90,CLT} = (1)(1.375 \text{ in.})(1,200 \text{ ksi})(12 \text{ in./ft}) + \frac{(2)(1.375 \text{ in.})(1,400 \text{ ksi})(12 \text{ in./ft})}{30} = 21,300 \text{ k/ft}$$

$$(EA)_{22,eff} = (EA)_{C,eff} + (EA)_{90,CLT} = 167,000 \text{ k/ft}$$

$$f22 = \frac{(EA)_{22,eff}}{(EA)_{22,SAP}} = 1.09$$

$$\bar{z}_{f22} = \frac{(EA)_{C,eff} * \bar{z}_C + (EA)_{90,CLT} * \bar{z}_{CLT}}{(EA)_{C,eff} + (EA)_{90,CLT}} = 6.04 \text{ in.}$$

Note that γ is applied to the concrete contribution, as opposed to the CLT contribution, for calculating f11, f22, \bar{z}_{f11} and \bar{z}_{f22} regardless of the position of the neutral axis of the CLT-concrete composite assembly. This is because the $(EA)_{11,eff}$ and $(EA)_{22,eff}$ components of the CLT-concrete assembly act compositely with the glulam beams. In this example, the CLT-concrete assembly is modeled with rigid end offsets from the glulam beams, which is equivalent to assuming 100% composite action at the interface between the glulam beam and CLT panel. For this combined assembly (glulam, CLT and concrete), it is reasonable to assume that the neutral axis would be below the bottom of the concrete topping and therefore γ would apply to the concrete contribution to combined flexural stiffness. For determining the appropriate application of γ for calculating m11 and m22, which in this example differs to that for f11 and f22, see Section 9.3.3.

9.3.2 Membrane f12 Modifier Derivation

Modifier f12 corresponds to in-plane shear stiffness. See Figure 9-4. This property is not likely to significantly impact the floor vibration performance. Therefore, while it is important that the value used be reasonable, it is not essential that it be exact.

$$(GA)_{SAP} = (700 \text{ ksi})\left(9\frac{1}{8} \text{ in.}\right)(12 \text{ in./ft}) = 76,700 \text{ k/ft}$$

$G_{CLT} = 30 \text{ ksi}$ per *CLT Horizontal Diaphragm Design Example* (Structurlam, 2015) or published CLT manufacturer data adjusted for splices when available

$$G_{conc} = 0.4E = 0.4 * 1.35 * 57\sqrt{f'_c} = 0.4 * 1.35 * 57\sqrt{4,000 \text{ psi}} = 1,950 \text{ ksi}$$

$$(GA)_{eff} = G_{CLT}A_{CLT} + G_{conc}A_{conc} = (30 \text{ ksi})\left(4\frac{1}{8} \text{ in.}\right)(12 \text{ in./ft}) + (1,950 \text{ ksi})(5 \text{ in.})(12 \text{ in./ft}) \\ = 118,300 \text{ k/ft}$$

$$f_{12} = \frac{(GA)_{eff}}{(GA)_{SAP}} = 1.54$$

9.3.3 Bending m11, m22, and m12 Modifier Derivation

Modifiers m11 and m22 correspond to flexural stiffness values for the shell local 1 and 2 faces, respectively. See Figure 9-4. The effective flexural stiffness of the partial CLT-concrete assembly is derived using the gamma method equations presented in Section 3.4.1. Since, from the calculations presented in Section 9.3.1, it was found that the neutral axis of the composite section is in the concrete topping (i.e., z_{f11}^- and z_{f22}^- both exceed the CLT thickness), γ is applied to the CLT rather than the concrete contribution (i.e., $\gamma_{CLT}=0.5$ and $\gamma_c=1.0$). Note that this occurs for assemblies with a large ratio of concrete to CLT thickness. In more typical floors for residential and office use, the neutral axis almost always falls within the CLT (i.e., $\gamma_{CLT}=1.0$ and $\gamma_c=0.5$).

In this example, it is assumed that the CLT panel effective stiffness values $(EI)_{0,CLT}$ and $(EI)_{90,CLT}$ are known values provided by either PRG 320-19 or the manufacturer. If these parameters are not known, they can be derived following the procedure outlined in the Appendix X3 of PRG 320-19.

$$(EI)_{11,SAP} = (EI)_{22,SAP} = E\frac{1}{12}t^3b = (1,400 \text{ ksi})\frac{1}{12}\left(9\frac{1}{8} \text{ in.}\right)^3 (12 \text{ in./ft}) = 1,064,000 \text{ k} * \text{in.}^2 / \text{ft}$$

$$(EI)_{0,CLT} = 95,000 \text{ k} * \text{in.}^2 / \text{ft} \text{ per PRG 320-19 Table A2}$$

$$(EA)_{0,CLT} = 46,900 \text{ k/ft} \text{ per Section 9.3.1}$$

$$t_{CLT} = 4.125 \text{ in.}$$

$$\gamma_{CLT} = 0.5$$

$$(EI)_C = 1.35 * 57\sqrt{4000 \text{ psi}} * \frac{1}{12} * 12 \text{ in./ft} * (5 \text{ in.})^3 = 608,300 \text{ k} * \text{in.}^2 / \text{ft}$$

$$(EA)_C = 1.35 * 57\sqrt{4000 \text{ psi}} * 12 \text{ in./ft} * 5 \text{ in.} = 292,000 \text{ k/ft}$$

$$t_c = 5 \text{ in.}$$

$$\gamma_c = 1.0$$

$$r = \frac{t_c}{2} + \frac{t_{CLT}}{2} = 4.5625 \text{ in.}$$

$$a_c = \frac{\gamma_{CLT} * (EA)_{0,CLT} * r}{\gamma_c * (EA)_C + \gamma_{CLT} * (EA)_{0,CLT}} = 0.34 \text{ in.}$$

$a_c < t_c/2$ which is in the concrete, so $\gamma_{CLT} < 1.0$ assumption confirmed

$$a_{CLT} = \frac{\gamma_C * (EA)_C * r}{\gamma_C * (EA)_C + \gamma_{CLT} * (EA)_{0,CLT}} = 4.22 \text{ in.}$$

or from geometry: $a_{CLT} = r - a_C = 4.22 \text{ in.}$

$$(EI)_{11,eff} = (EI)_C + (EI)_{0,CLT} + \gamma_C * (EA)_C * a_C^2 + \gamma_{CLT} * (EA)_{0,CLT} * a_{CLT}^2 = 1,155,000 \text{ k} * \text{in.}^2 / \text{ft}$$

$$m11 = \frac{(EI)_{11,eff}}{(EI)_{11,SAP}} = 1.09$$

Repeating the derivation process for the weak axis direction:

$$(EI)_{22,SAP} = (EI)_{11,SAP} = 1,064,000 \text{ k} * \text{in.}^2 / \text{ft}$$

$$(EI)_{90,CLT} = 3,100 \text{ k} * \text{in.}^2 / \text{ft} \text{ per PRG 320-19 Table A2}$$

$$(EA)_{90,CLT} = 21,300 \text{ k/ft} \text{ per Section 9.3.1}$$

$$\gamma_{CLT} = 0.5$$

$$(EI)_C = 608,300 \text{ k} * \text{in.}^2 / \text{ft}$$

$$(EA)_C = 292,000 \text{ k/ft}$$

$$\gamma_C = 1.0$$

$$a_C = \frac{\gamma_{CLT} * (EA)_{90,CLT} * r}{\gamma_C * (EA)_C + \gamma_{CLT} * (EA)_{90,CLT}} = 0.16 \text{ in.}$$

$a_C < t_C/2$ which is in the concrete, so $\gamma_{CLT} < 1.0$ assumption confirmed

$$a_{CLT} = r - a_C = 4.40 \text{ in.}$$

$$(EI)_{22,eff} = (EI)_C + (EI)_{90,CLT} + \gamma_C * (EA)_C * a_C^2 + \gamma_{CLT} * (EA)_{90,CLT} * a_{CLT}^2 = 826,000 \text{ k} * \text{in.}^2 / \text{ft}$$

$$m22 = \frac{(EI)_{22,eff}}{(EI)_{22,SAP}} = 0.78$$

The m12 modifier corresponds to the shell torsional stiffness. See Figure 9-4. There are not well known published values for this parameter. It is the opinion of the authors that a reasonable value for m12 likely falls between that for v13 and v23 calculated in Section 9.3.4. In this example the lower of the two values has been chosen. If a floor framing system is likely to depend significantly on the torsional properties, upper- and lower-bound analyses setting m12 equal to v13 and v23 may be necessary.

$$m12 = \min(v13, v23) = 1.53$$

9.3.4 Shear V13 and V23 Modifier Derivation

Modifiers v13 and v23 correspond to out-of-plane shear stiffness values for the shell local 1 and 2 faces respectively. See Figure 9-4.

$$(GA)_{13,SAP} = (GA)_{23,SAP} = (700 \text{ ksi})\left(9\frac{1}{8} \text{ in.}\right)(12 \text{ in./ft}) = 76,700 \text{ k/ft}$$

$$(GA)_{0,eff} = 460 \text{ k/ft per PRG 320-19 Table A2}$$

$$(GA)_{13,eff} = (GA)_{0,eff} + G_{Conc}t_{Conc}b = (460 \text{ k/ft}) + (1,950 \text{ ksi})(5 \text{ in.})(12 \text{ in./ft}) = 117,500 \text{ k/ft}$$

$$v13 = \frac{(GA)_{13,eff}}{(GA)_{13,SAP}} = 1.53$$

$$(GA)_{90,eff} = 520 \text{ k/ft per PRG 320-19 Table A2}$$

$$(GA)_{23,eff} = (GA)_{90,eff} + G_{Conc}t_{Conc}b = (520 \text{ k/ft}) + (1,950 \text{ ksi})(5 \text{ in.})(12 \text{ in./ft}) = 117,500 \text{ k/ft}$$

$$v23 = \frac{(GA)_{23,eff}}{(GA)_{23,SAP}} = 1.53$$

9.3.5 Mass and Weight Modifier Derivation

In order for the modal analysis to be accurate, it is important to properly define the mass within the system. The mass and weight modifiers, which SAP2000 uses to calculate self-mass of modeled elements, will account for the CLT and concrete topping weight; all other masses will be included as a superimposed load as described in the following section.

$$\text{Mass Modifier} = \text{Weight Modifier} = \frac{\text{Actual Weight}}{\text{SAP2000 Calc Weight}}$$

$$\text{SAP2000 Calc Weight} = t_{slab}W_{CLT} = \left(9\frac{1}{8} \text{ in.}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)(26 \text{ lb/ft}^3) = 19.8 \text{ lb/ft}^2$$

$$\begin{aligned} \text{Actual Weight} &= t_{CLT}W_{CLT} + t_{Conc}W_{Conc} = \left(4\frac{1}{8} \text{ in.}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)(26 \text{ lb/ft}^3) + (5 \text{ in.})\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right)(150 \text{ lb/ft}^3) \\ &= 71.4 \text{ lb/ft}^2 \end{aligned}$$

$$\text{Mass Modifier} = \text{Weight Modifier} = 3.6$$

9.4 Loading/Mass

The masses for the modeled glulam frame, CLT and concrete topping are defined in SAP2000 under the dead load case. As previously described, all additional mass is included as a uniform shell superimposed dead load, which also includes a portion of the floor live loading (Section 3.1). See Table 9-3 and Figure 9-9.

TABLE 9-7: Superimposed dead load takeoff

Mechanical/electrical/plumbing	4 psf
Flooring	2 psf
Portion of live load for vibration analysis	10 psf
Total	16 psf

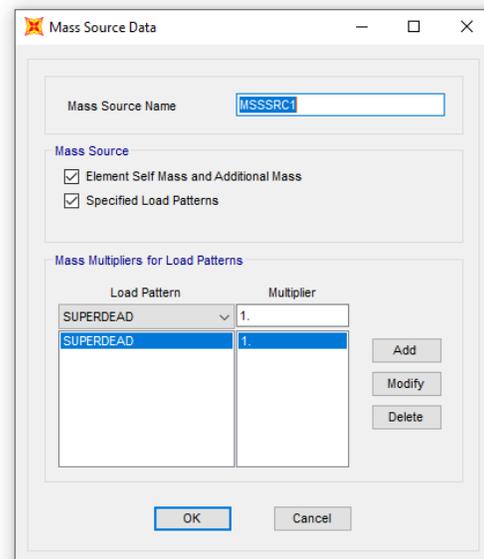


FIGURE 9-9: Mass source data input

9.5 Shell Meshing

The mesh should be fine enough to capture the floor plate behavior but not so refined as to slow computation. The method for meshing the floor shell element is up to the analyst but should be coordinated with the node locations required for post-processing (Section 5.8). For this example, a 24-in. square mesh is appropriate for the framing geometry. See Figure 9-10. This element size allows for the meshing geometry to align with beam framing in a regular pattern and has been easily accomplished using the “Edit → Divide Areas” tool in SAP2000.

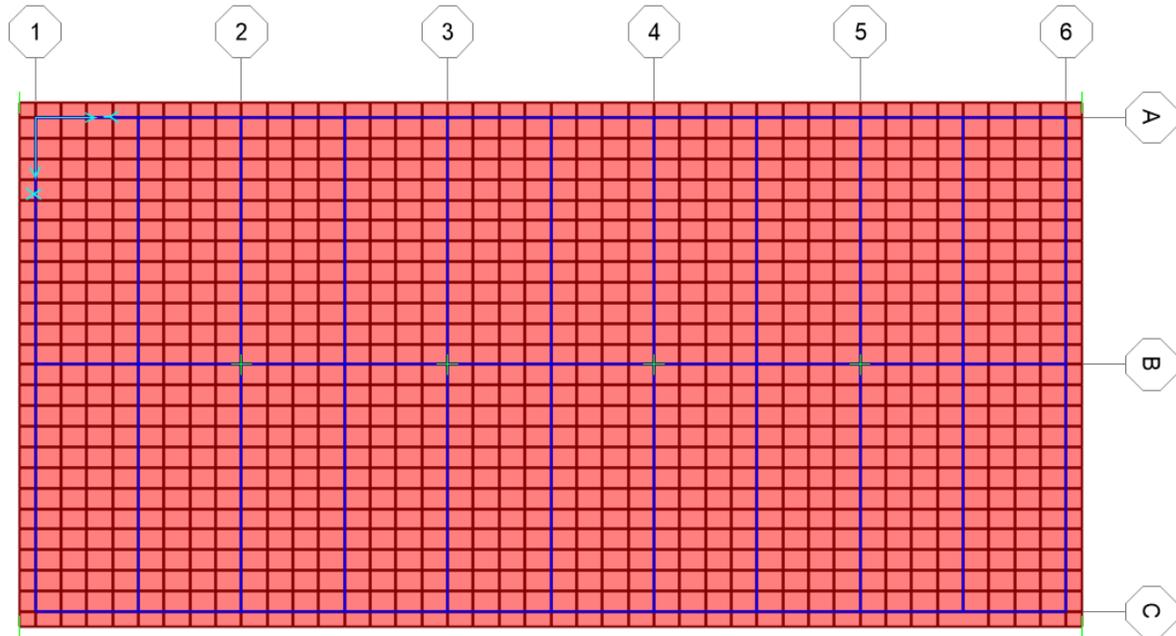


FIGURE 9-10: Floor plate shell meshing

9.6 Modal Analysis Output

The modal load case should be defined to find an appropriate number of modes to fully capture the behavior for floor vibration. For transient response analysis (Section 4.3.2), it is recommended to solve for all modes of the structure with a frequency less than twice the first fundamental natural frequency. For this example, the first 22 modes are produced by the model with a range of frequencies between 14 Hz and 30 Hz. Since 14 Hz exceeds four times the fastest walking frequency anticipated, only transient response analysis need be checked. Figure 9-11 shows the first eight mode shapes produced by SAP2000 along with the corresponding frequency. It should be noted that SAP2000 assigns a constant modal mass of $1 \text{ kip-in.}\cdot\text{s}^2 = 83.33 \text{ lb-ft}\cdot\text{s}^2$ for all modes and weighting of mode shapes is achieved through the joint displacement values.

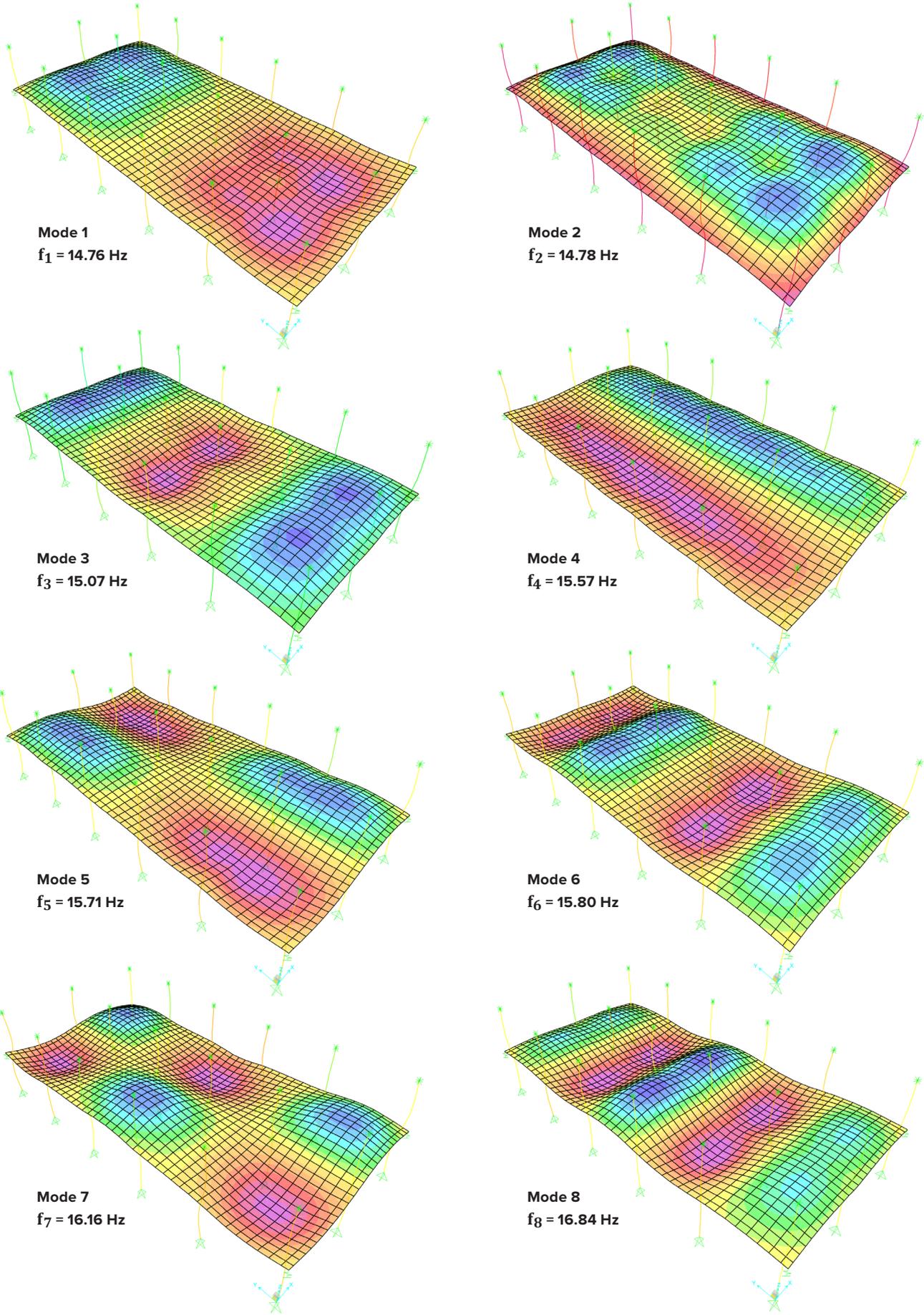


FIGURE 9-11: SAP2000 modal analysis results for the first eight modes

9.7 Post-Processing and Model Results

The modal participation factors and joint displacements for the nodes of interest are then exported. Output tables can be exported from SAP2000 to another program for subsequent calculations following the transient response analysis method presented in Section 4.3.2. In this example, an Excel spreadsheet is used.

It is important to select a variety of node locations across the floor plate to fully understand the behavior of the system. While the center point of a bay is likely to exhibit the largest vibration response, checking points along beam and girder lines for both interior and exterior bays can provide feedback as to which elements are contributing most to the excitation. This, along with the mode shapes, indicate which elements can be most effectively adjusted to tune the floor system to satisfy the desired vibration criteria.

The following are example calculations for point 1820, which occurs in a corner bay. See Figure 9-13. For simplicity in the calculation procedure, the excitation and response nodes have been set to the same location. In practice, this is often conservative as the excitation location is likely to be at least some distance from the sensitive equipment. As noted in the introduction, due to the high-frequency nature of the floor, the resonant response from walker excitation will not control over the transient response and is therefore neglected.

Step 1: Establish appropriate input parameters:

Walking frequency of interest, $f_w = 2.0$ Hz

Weight of walker, $P = 168$ lb

Ratio of critical damping, $\zeta = 0.035$

Step 2: Export the SAP2000 modal output parameters f_m , $\mu_{e,m}$, $\mu_{r,m}$, and \hat{m}_m for the node(s) of interest.

Step 3: Calculate the effective footfall impulse loads, $I_{eff,m}$, and then peak velocity, \hat{v}_m , for each mode. This procedure is shown in Table 9-4. Note that SAP2000 uses consistent units for $\mu_{e,m}$, $\mu_{r,m}$, and \hat{m}_m which correctly cancel in the calculations even if lb-ft-s² is not a mass unit per se.

TABLE 9-11: Modal post-processing for $I_{eff,m}$ and \hat{v}_m

Mode	f_m	f_w	$I_{eff,m} = \frac{P * f_w^{1.43}}{17.8 * f_m^{1.30}}$	$\mu_{e,m}$	$\mu_{r,m}$	\hat{m}_m	$\hat{v}_m = \mu_{e,m} \mu_{r,m} \frac{I_{eff,m}}{\hat{m}_m}$
	Hz	Hz	lb-s	ft	ft	lb-ft-s ²	ft/s
1	14.76	2.0	0.769	-0.1252	-0.1252	83.33	1.45E-04
2	14.78	2.0	0.767	-0.1140	-0.1140	83.33	1.20E-04
3	15.07	2.0	0.748	0.0877	0.0877	83.33	6.90E-05
4	15.57	2.0	0.717	0.0941	0.0941	83.33	7.62E-05
5	15.71	2.0	0.708	0.1375	0.1375	83.33	1.61E-04
6	15.80	2.0	0.703	0.0729	0.0729	83.33	4.49E-05
7	16.16	2.0	0.683	-0.1236	-0.1236	83.33	1.25E-04
8	16.84	2.0	0.648	0.0345	0.0345	83.33	9.26E-06
9	16.91	2.0	0.644	-0.0779	-0.0779	83.33	4.69E-05
10	17.96	2.0	0.595	-0.0204	-0.0204	83.33	2.98E-06
11	21.53	2.0	0.470	0.0922	0.0922	83.33	4.79E-05
12	22.86	2.0	0.435	-0.1364	-0.1364	83.33	9.71E-05
13	23.43	2.0	0.421	0.0841	0.0841	83.33	3.57E-05
14	24.77	2.0	0.392	-0.1314	-0.1314	83.33	8.12E-05
15	25.19	2.0	0.384	0.1183	0.1183	83.33	6.44E-05
16	26.33	2.0	0.362	0.0213	0.0213	83.33	1.96E-06
17	27.06	2.0	0.349	-0.1263	-0.1263	83.33	6.69E-05
18	27.95	2.0	0.335	-0.0606	-0.0606	83.33	1.47E-05
19	28.77	2.0	0.323	0.0107	0.0107	83.33	4.41E-07
20	28.83	2.0	0.322	0.0228	0.0228	83.33	2.01E-06
21	29.32	2.0	0.315	0.0060	0.0060	83.33	1.37E-07
22	30.01	2.0	0.306	0.0743	0.0743	83.33	2.02E-05

Step 4: Establish the velocity response for each mode over the period of one footfall, T_w , using the equations below. The velocity response functions for the first eight mode shapes are graphed in Figure 9-12 for reference.

$$v_m(t) = \hat{v}_m e^{-2\pi\zeta_f t} \sin(2\pi f_m t)$$

$$T_w = \frac{1}{f_w}$$

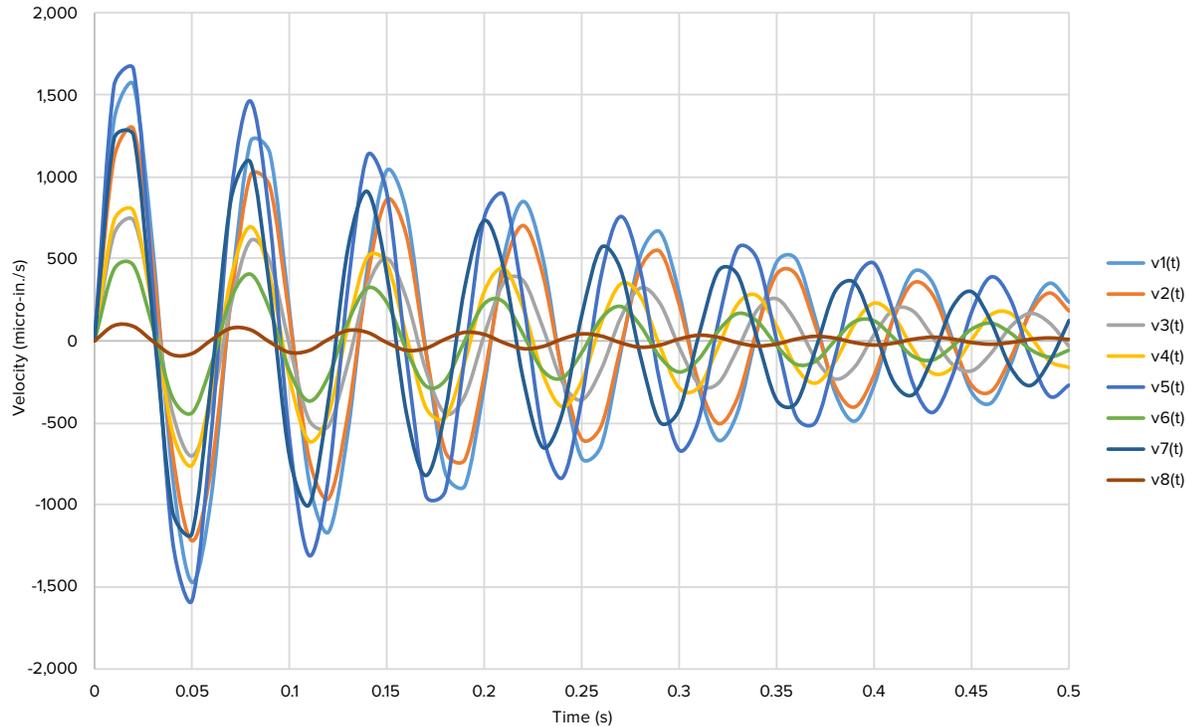


FIGURE 9-12: Velocity response over one footfall period for the first eight modes

Step 5: Determine the total velocity response function to each footfall, $v(t)$, by summing all individual mode ($N = 22$) velocity response functions.

$$v(t) = \sum_{m=1}^N v_m(t)$$

Step 6: Calculate the resulting root-mean-square velocity, V_{RMS} , over the period of one footfall using the following equation. Note that no frequency weighting of the velocity is applied in this example. All frequencies exceed 8 Hz and therefore the weighting function is 1.0 per Section 4.3.2.

$$V_{RMS} = \sqrt{\frac{1}{T_w} \int_0^{T_w} v(t)^2 dt}$$

$$V_{RMS} = 3,300 \text{ micro-in./s}$$

Sensitive equipment velocity criteria curves are typically presented in terms of one-third octave band RMS velocity, $V_{RMS,1/3}$, rather than total RMS velocity, V_{RMS} . As described in Section 4.3.2, $V_{RMS,1/3}$ can be calculated by performing the RMS calculation here except using a revised $v(t)$ where only the modes within the one-third

octave band are included in Step 5. For this example, the largest value of $V_{RMS,1/3}$ over all one-third octave bands occurs at a one-third octave center frequency of 16 Hz (including modes between 14.3 Hz and 18 Hz).

$$V_{RMS,1/3} = 3,100 \text{ micro-in./s}$$

Step 7: Repeat this process for every arrangement of excitation and response nodes across the floor plate of interest to the analyst.

Figure 9-13 below shows a plan view of the floor plate with several critical points of interest that have been evaluated using the transient response analysis method. This calculation process is repeated for each point of interest and the results are presented in Table 9-5.



FIGURE 9-13: Selected points of interest

TABLE 9-5: Floor RMS velocities at selected points of interest

Point Label	Total RMS Velocity (mips)	Maximum 1/3- Octave Band RMS Velocity (mips)	Notes
65	1,300	1,200	Mid-span of interior girder, center bay
67	800	600	Mid-span of perimeter girder, center bay
71	1,600	1,500	Mid-span of interior girder, end bay
73	1,200	1,000	Mid-span of perimeter girder, end bay
1814	2,700	2,200	Mid-span of perimeter beam
1337	3,600	3,500	Mid-span of beam, end bay
1820	3,300	3,100	Mid-span of slab, end bay
1838	2,100	1,800	Mid-span of interior beam, grid line
1359	3,100	2,800	Mid-span of slab, interior bay
1349	2,100	1,800	Mid-span of interior beam, grid line

9.8 Example Conclusion

As seen in Table 9-5, all evaluated points on the floor plate are below the 4,000 micro-in./s RMS velocity criterion.

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