Vibration Serviceability and Dynamic Modeling of Cold-Formed Steel Floor Systems

by

Russell A. Parnell

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of this thesis, including any required final revisions, as accepted by my examiners.

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Abstract

The use of cold-formed steel as a framework for floor systems in multi-story buildings and single occupancy residences is becoming an increasingly popular alternative to traditional materials and techniques. Builders and designers have recognized that the high strength-to-weight ratio provided by the cross-section of cold-formed steel members permits lighter structures and longer spans. The longer spans and lighter structures associated with cold-formed steel floor systems can result in vibration serviceability issues if proper design considerations are not made. Providing sufficient damping within the structure is the most effective way to ensure that occupants are comfortable under typical residential and office service loads. The modern, open-concept interior has open floor plans with few partitions and long spans, which result in inherently low structural damping. Cold-formed steel floor systems also have less mass than traditional floor systems, which will increase the amplitude of acceleration response.

The vibration problems that may be present in cold-formed steel floor systems, like any other floor system, can be addressed if proper consideration is given by designers. Traditional design approaches for vibration serviceability have proven inadequate, and there are no current methods available to designers for calculating the response of cold-formed steel floor systems. In order to design a floor system to properly address occupant comfort, consideration must be given for the type of dynamic loading, resonance, dynamic response, and stiffness of the floor system. The objective of this thesis is to improve the understanding of the dynamic characteristics of cold-formed steel floor systems, and recommend an adequate model for predicting the dynamic response and modal properties of floor systems, in order to aid the design process.

This thesis presents the results of an extensive laboratory and field study on the vibration of cold-formed steel floor systems. Floor systems built with cold-formed steel TreadyReady® joists and subfloor assemblies containing OSB, FORTACRETE®, sound reduction board, cold-formed steel deck, and LEVELROCK® topping were examined. Previous research has presented the observed influence of construction details on the modal properties of the laboratory floor systems tested. This thesis discusses the influence of different details on the transverse stiffness of the floor systems. It was found that effectively restrained strongbacks, and cold-formed steel deck subfloor assemblies provided significant increases in transverse stiffness. Based on the analysis of the field testing data, recommended design damping ratios are provided for floor systems constructed with the materials investigated in this study.

Floor response that can be compared to serviceability criteria is presented. The peak RMS acceleration from walking excitation was found to be within the acceptable range for the ISO criterion based on residential occupancy, and the static deflection from a 1 kN point load was found to be within the acceptable range of Onysko's criterion. An adequate design criterion for vibration requires a limiting value, and a means of estimating floor response for comparison. The AISC, ATC, and Smith, Chui, and Hu Orthotropic Plate design methods were evaluated by comparing predicted frequency against measured frequency for the test floors. The ATC and Smith, Chui, and Hu Orthotropic Plate methods were evaluated by comparing predicted deflection against measured deflection for the test floors. The ATC method is recommended as the best method for calculating floor response based on current publications.

A design procedure is recommended for cold-formed steel floor systems, using the ATC design guide. The ATC acceleration criterion for walking excitation must be met for floors with fundamental frequencies of less than 15 Hz, and the ATC static deflection criterion must be met for all floors. Proposed modifications to the ATC method to improve the design of cold-formed steel floors include: adopting the recommended design damping ratios from this thesis; adopting the frequency-weighted ISO limiting acceleration and, obtaining several coefficients and empirical expressions that are relevant to cold-formed steel floors from further testing. Recommendations for improving the floor testing procedures at the University of Waterloo are given.

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Chapter 1. Introduction and Background

1.1 Introduction

The use of cold-formed steel, and other lightweight components, as a framework for floor systems in multi-story buildings and single occupancy residences is becoming an increasingly popular alternative to traditional materials and techniques. Builders and designers have recognized that the high strength-to-weight ratio provided by the cross-section of cold-formed steel members allows lighter structures and longer spans. Cold-formed steel joists do not require specialty trades for installation. Floor systems built on the cold-formed steel joist framework can be built quickly and precisely, are impervious to rot and insect damage and can have long fire-separation ratings when designed appropriately. The floor systems are ideal for open-concept residential and commercial floor plans, with allowable clear span lengths of 10 m or greater. If joists with large lip-reinforced openings are used, mechanical components can be installed within the depth of the joists, and the overall inter-story depth is reduced. For these reasons, cold-formed steel floor systems can be advantageous when used in place of light-frame floor systems, built with wood members, and heavy floor systems, built with reinforced concrete and hot-rolled steel joists.

The structural and cost benefits associated with cold-formed steel floor systems can result in vibration serviceability issues if proper design considerations are not made. Providing sufficient damping within the structure is the most effective way to ensure that occupants are comfortable under typical residential and office service loads (Lenzen, 1966). The modern, open-concept interior that is made possible by cold-formed steel joists results in inherently low structural damping because it does not have the damping contributions from partition walls and heavy furnishings (Hanagan, 2005). Cold-formed steel floor systems are stiffer than similar light-frame wood floors, which also reduces the structural damping of the system. Cold-formed steel floor systems are lighter than equivalent floor systems with hot-rolled or open-web steel joists and concrete slabs, which is advantageous for structural design and overall building cost. However, less mass will increase the amplitude of the response to dynamic excitation.

The vibration problems that may be present in cold-formed steel floor systems, like any other floor system, can be addressed if proper consideration is given by designers. Traditional floor serviceability requirements, like those set by the National Association of Home Builders (NAHB) in the United States, limit the deflection of a floor under a uniformly distributed service load to a fraction of span length. The NAHB limit is 1/480. Extensive studies have shown that the traditional serviceability limit of 1/360 does not result in satisfactory occupant perception of vibration (Onysko, 1985). In order to design a floor system to properly address occupant comfort, consideration must be given for the type of dynamic loading, resonance, dynamic response, and stiffness of the floor system. With an understanding of the dynamic characteristics of cold-formed steel floor systems and an adequate model to predict the dynamic response and modal properties, these floor systems can be designed to meet occupant requirements.

1.2 Description of Problem

There is a need to expand on the previous testing of cold-formed steel floor systems, particularly in terms of the vibration serviceability of in situ floor systems. Extensive studies have been conducted on floor vibration, but few have been based on floors with a cold-formed steel structural framework. Past studies of cold-formed steel floor systems have focused on laboratory specimens. Testing in situ floor systems will provide quantifiable performance characteristics of actual in-service floors, which can be compared to accepted vibration serviceability design criteria.

There is also a need to provide a means for cold-formed steel floor systems to be designed to meet the applicable vibration serviceability design criteria. There are several design procedures available for floor vibration, but they have been developed for either light-frame timber-based systems or heavy steel and concrete systems. They contain equations that are based on structural behavior of specific materials, and require several coefficients for empirical expressions which have been derived from materials testing. Evaluating the characteristics of floor systems in both laboratory and in situ conditions using current design methods will provide an estimate on the degree of accuracy they provide, and indicate areas where the models can be modified and improved for this particular use.

1.3 Objectives

This thesis contains the comprehensive results of a floor vibration study conducted by the Canadian Cold-Formed Steel Research Group at the University of Waterloo. The study consisted of an extensive set of laboratory specimens and several in situ floors located throughout the United States. Two graduate research assistants participated in the study, and this thesis is the second to be produced from the study (Davis, 2008). Davis concentrates on the laboratory portion of the study, and the influence of varying construction details on the modal properties of the floor systems. This thesis will expand on specific aspects of the observed behavior of construction details, and focus on the measured responses and properties of the laboratory and in situ floor systems which are relevant to vibration serviceability performance and design.

The objectives of the research presented in this thesis are to:

- investigate the influence of construction details on the transverse stiffness of floor systems, using frequency separation and static deflection for comparison;
- recommend a damping ratio specific to the design of cold-formed steel joist floor systems, accounting for subfloor configurations, which can be used in the prediction of dynamic response;
- evaluate the vibration serviceability performance of the in situ floor systems, using the response from both impulse and walking excitation, as well as static deflection, as the quantities compared against acceptability criterion;
- evaluate the accuracy and applicability of three commonly used design methods for predicting fundamental frequency, static deflection, and acceleration response to walking excitation;
- develop a design procedure for cold-formed steel joist floor systems that is based on current, published design methods, and to recommend modifications to improve applicability and accuracy for cold-formed steel joist floor systems; and,
- recommend improvements for equipment and procedures for future testing of material-specific coefficients and empirical expressions used by the Canadian Cold-Formed Steel Research Group at the University of Waterloo.

1.4 Floor Vibration: Background and Terminology

This section will present some of the terminology and concepts, specific to structural dynamics and floor vibration, which are used throughout this thesis. The terminology can be divided into two categories: terms describing vibration, and terms describing the relevant properties of floor systems.

1.4.1 Terms Describing Vibration

In a general sense, the motion associated with vibration is described using terminology that is applied to repetitive wave forms: *period*, *frequency*, *wavelength*, and *amplitude*. *Period (T)* is the amount of time taken to complete one cycle, in seconds. *Frequency (f)* is the number of cycles per second, in Hz, and is the inverse of the period. *Wavelength* is the horizontal distance between the same point on two successive cycles of the wave. *Amplitude* is the vertical distance between zero and maximum displacement of the wave form.

Floor vibration is the time-varying response of a floor system due some type of *dynamic excitation*. It is commonly quantified as the displacement, velocity or acceleration at a point on the floor system, as a function of time. Typical excitations involve a mechanism which induces deflection of the floor; and can be either *harmonic* or *transient* in nature. *Harmonic excitation* is a continuous process that excites the floor with a combination of one or more definite forcing frequencies, known as *harmonics*.

Transient vibration is a singular process that excites the floor with discrete impulses, followed by *free vibration*. Harmonic excitation is typically produced by rotating machinery. Transient excitation can be produced by human activity on a floor system. *Walking excitation* is a combination of both harmonic loading and impulses, so both aspects must be taken into account for design. Human activity, such as walking, jumping, and dancing, can result in harmonic and transient excitation of a floor system (Murray, Allen, & Ungar, 1997). *Walking excitation* consists of regular, discrete impulses that cycle with harmonics of approximately 2 Hz, 4 Hz, 6 Hz, and 8 Hz.

The vibration response of a floor system can consist of both a *steady-state*, and a *transient* component. The *steady-state* response occurs during harmonic excitation. The *transient* response occurs during *free vibration*, when no excitation is occurring, and typically decays to zero amplitude over the *period of response* due to *damping*. The nature of the vibration response of the floor is a function of both the floor's *modal properties* and the excitation.

1.4.2 Terms Describing the Properties of Floor Systems

The properties of a structural system that define the dynamic characteristics of that system are known as *modal properties*. The key modal properties for floor vibration are *natural frequency*, *mode shape*, and *modal damping ratio*. A structural system will have a unique set of modal properties for each degree of freedom, or *vibration mode*. Although floor systems are continuous structures, the vibration response can be sufficiently characterized using the first few vibration modes due to the nature of residential floor systems under occupant-induced excitation.

The cycling rate of the free vibration response for each vibration mode is the *natural frequency* of that mode. The natural frequency for a the first vibration mode of structure is the *fundamental frequency*. Many of the models used to predict floor vibration response consist of one degree of freedom, and consider only the fundamental frequency. The *mode shape* is a unique expression of the relative position of all points on the structure during free vibration for each mode of vibration. The mode shapes and natural frequencies of floor systems are governed by the mass of the system, and cross-sectional properties of the floor in both the primary joist and transverse directions, and the shape and dimensions of the floor plan.

The *modal damping ratio* defines the rate of decay of the peak amplitudes of the free vibration response for each vibration mode. It is expressed as a percentage of the *critical damping ratio*, which is defined as the amount of damping required to stop motion after exactly one cycle of free vibration. Several types of damping can exist with in a structure, such as *viscous damping*, *coulomb damping* and *hysteretic damping*. *Viscous damping* is decay which is proportional to the velocity of the system, and is the damping mechanism of hydraulic shock absorbers. *Coulomb damping* is decay which occurs through sliding friction. *Hysteretic damping*, also know as *structural damping*, is non-linear decay mechanism which considers relative displacement of structural components. While hysteretic damping is most applicable for vibration of structures, it is often too complex for the simple design models employed for floor vibration. Viscous damping is commonly used in literature, and is expressed as a percentage of the critical damping value for the structure. Values ranging from 1% to 12% are suggested by various authors.

Generally, damping is affected by member connections, partitions, ceiling details and heavier furnishings (Murray, Allen, & Ungar, 1997). Damping is an inherent property of the structure of the floor and any attached non-structural elements. The structure itself exhibits both material damping (viscous), and joint and connection damping (coulomb). Non-structural elements can have significant contributions to the overall damping of the floor system based on their mass, material and geometry (Xin, 1996). Because almost all floor systems are unique in terms of non-structural elements, it is very difficult to predict overall damping ratios without measurements taken from a similar floor system.

There is no reliable method for calculating the damping properties of a floor system due to complexity and construction variation. It has been found that experimentally measured damping ratios are inconsistent for different test methods. The heel drop test produces higher damping values than other methods; the person exciting the floor can make a significant contribution to the system's damping (Rainer & Pernica, 1981).

1.4.3 SDoF Beam Model

The single degree of freedom (SDoF) spring-dashpot system (Figure 1-1) can be used to illustrate the how the excitation and modal properties can be combined to estimate the response of a floor system. This is a common vibration model used to estimate the modal properties and response of floor systems for design purposes. The equation of motion for this system is shown in the following equation:

$$m\ddot{x} + c\dot{x} + kx = F(t) \tag{1-1}$$

where:

m is the mass in motion;

c is the viscous damping coefficient for the system;

k is spring stiffness;

$F(t)$ is an arbitrary forcing function (of time); and,

x is displacement (with associated derivatives of velocity, $\dot{x}$, and acceleration, $\ddot{x}$).

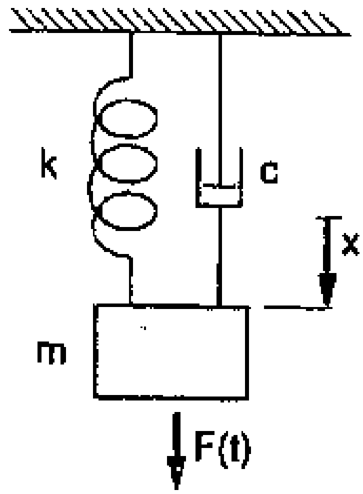

Figure 1-1: Single Degree of Freedom System (Xin, 1996)

The dynamic response depends on the forcing function and the system properties. The fundamental frequency and mode shape are determined from the eigenvalue and eigenvector used in the solution of Eq. 1-1. Assumptions must be made for the structural system and member interactions floor in order to idealize the floor properties to suit the form of this system.

The response of a floor system to a harmonic load depends on the forcing frequency. In order to calculate the response of a floor system, the forcing function can be expressed as a Fourier series of the form:

$$F(t) = \sum_{i=1}^{\infty} P_o \alpha_i \sin{(2\pi f_i t - \varphi_i)} \tag{1-2}$$

where:

P_o is a constant excitation force;

α_i is the dynamic coefficient for the ith harmonic;

f_i is the forcing frequency for the ith harmonic; and,

φ_i is the phase lag of the ith harmonic, relative to the 1st harmonic.

A low forcing frequency, relative to the system's natural frequency will generate a response similar to static loading. A high forcing frequency will generate a response which is less than the static response. When the forcing frequency approaches the natural frequency of the system, resonance will occur, amplifying the load. This phenomenon is illustrated in Figure 1-2.

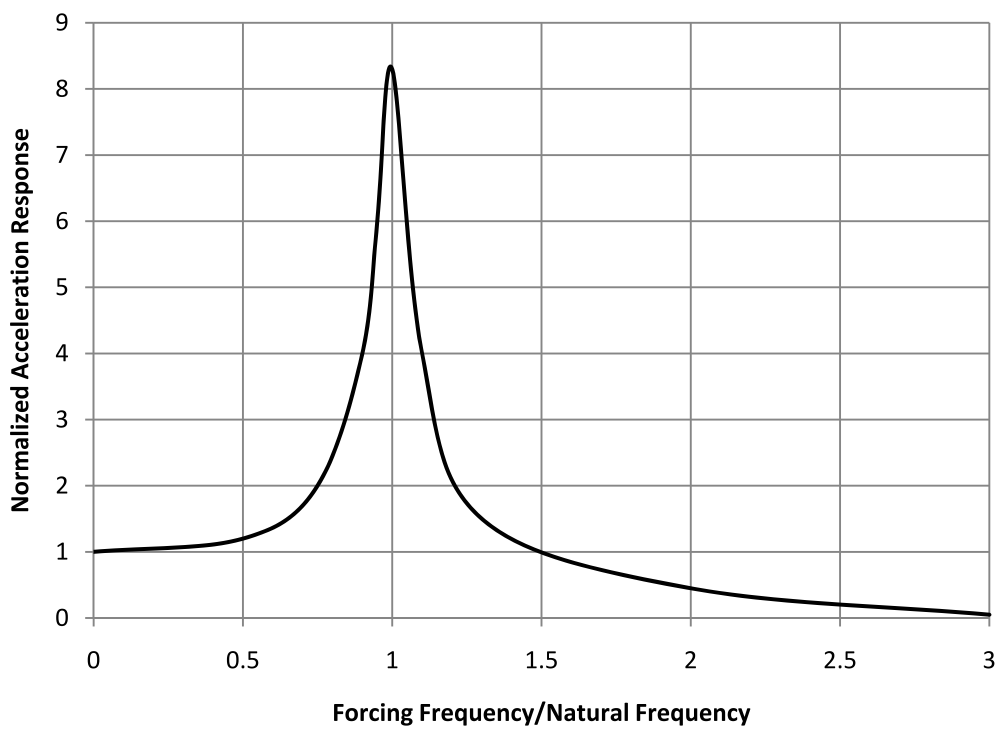

Figure 1-2: Resonance

Typically, the SDoF model is used to allow designers to estimate the fundamental frequency and peak acceleration response of a floor system, in order to evaluate the floor using vibration serviceability design criteria.

Chapter 2. Literature Review

This Chapter includes a review and discussion of several key publications for floor vibration performance and occupant comfort issues. Analysis procedures for floor vibration have two components: a human tolerance criterion and a method to predict the response of the floor system (Murray T. M., 1998). In order to provide a context for floor performance, human perception of vibration is discussed. Several relevant floor testing studies are reviewed, and the test methodologies and major results are presented. Finally, various design methods and criteria, which have been developed from floor testing studies or other research are discussed, and assessed in terms of their relevance to lightweight residential floor systems.

2.1 Perception

Human perception of vibration is a combination of movement of the floor system, physical perception, and the psychological reaction to vibration. The body position of the receiver, the activity of the receiver, and reradiated noise (e.g. china rattling in a cabinet) all effect the perception of vibration (ISO, 1989). The classification of uncomfortable vibration is subjective. Occupants in different settings will be more sensitive to the exact same vibration response than others, depending on frequency of occurrence, duration, and time of day. Humans are most sensitive to vibrations frequencies in the range of 2 Hz to 8 Hz due to resonance within the body cavity itself (Grether, 1971).

Occupant sensitivity depends on the activity of that person at the time of the perceived event. Individuals who are sleeping or sedentary will be more sensitive than those walking, running, dancing or doing aerobics. However, such activities may excite the floor in a harmonic or transient manner. The limiting values for acceleration in a specific occupancy are suggested to be related to the most sensitive occupancy of the building (Ellingwood & Tallin, 1984).

2.1.1 Steady State and Transient Excitation

Early research on occupant response to shock and vibration was published by Reiher and Meister in 1931. Testing was performed by applying a steady-state vibration to several individuals situated on floor systems, in various orientations. Their key finding was that sensitivity to vibration decreases as the excitation frequency increases. The Reiher-Meister scale, a design criterion rating acceptability based on floor frequency and vibration amplitude, was produced based on their results (Reiher & Meister, 1931).

Lenzen recognized that floor vibrations from typical use are transient in nature. Their rate of decay, along with frequency and amplitude, influences occupant comfort. Testing was conducted using transient vibrations in a similar fashion to the testing conducted by Reiher and Meister (1931). The original Reiher-Meister criterion was made less-stringent by a factor of 10 to account for the nature of human perception for transient excitations. This updated criterion is known as the Modified Reiher-Meister Scale.

Lenzen's observations provide a basis for understanding why several different types of models for floor vibration have been adopted. Long, heavy floors with low damping use a vibration model which assumes a cyclic response, and look to avoid resonant amplification. These models predict a response that is sensitive to fundamental frequency. Short, lightweight floors use a vibration model that considers initial displacement and local stiffness, not vibration, as higher inherent damping is assumed to cause significant decay of the response to excitation. These models predict a response that is sensitive to static stiffness. Lenzen's acceptability criterion is a reasonable modification of the Reiher-Meister scale, but it is a very simple model for predicting where a floor falls on the range of perception. The influence of damping is also not included in the criterion, which appears to contradict his conclusion that damping is the main factor influencing perceptibility. It is only applicable to floor spans of 7.32 m (24 ft) or less.

Wiss and Parmalee conducted experiments to rate the response of occupants to transient vibrations by varying frequencies, amplitudes and damping within the expected range generated by walking excitation on building floors. A significant conclusion is that vibration with the same peak amplitude and frequency becomes more tolerable as damping ratio is increased. This suggests the importance of the period of vibration decay, and that excitation amplitudes are 10 times less severe if the vibration is damped within 5 – 10 cycles (Wiss & Parmalee, 1974). These results support Lenzen's conclusions, and appear to discern a quantifiable difference between transient and harmonic excitation.

2.1.2 ISO Limit for Perceived Acceleration

The International Standards Organization (ISO) developed a limiting criterion for floor vibration, based on a maximum acceptable RMS acceleration for a given fundamental frequency of a structure, as a part of the ISO 2631 Standard for Mechanical Vibration and Shock – Evaluation of Human Exposure to Whole-body Vibration (ISO, 1989). The specific criterion applicable to floor vibration serviceability provides a limiting RMS acceleration for all fundamental frequencies as a baseline curve. Multipliers are introduced to account for the variation of occupant sensitivity and event frequency in different types of occupancies. For example: a multiplier of 10 is applied to the baseline curve for offices and residences, where occupants are sedentary; while a multiplier of 30 is used for shopping malls, where the occupants are typically moving and less sensitive (ISO, 1989). The vertical axis of the plot is intended to be RMS acceleration values, according to the ISO references, but is labeled peak or maximum acceleration in many sources. The shape of the baseline curve indicates the lowest tolerable acceleration levels are from 4 Hz to 8 Hz. This is for two reasons: human physiology makes occupants more sensitive in the 4 Hz to 8 Hz range (Grether, 1971); and, walking vibrations contain harmonics of 4, 6 and 8 Hz, which will lead to more frequent resonant events in this range (Allen, Onysko, & Murray, 1999).

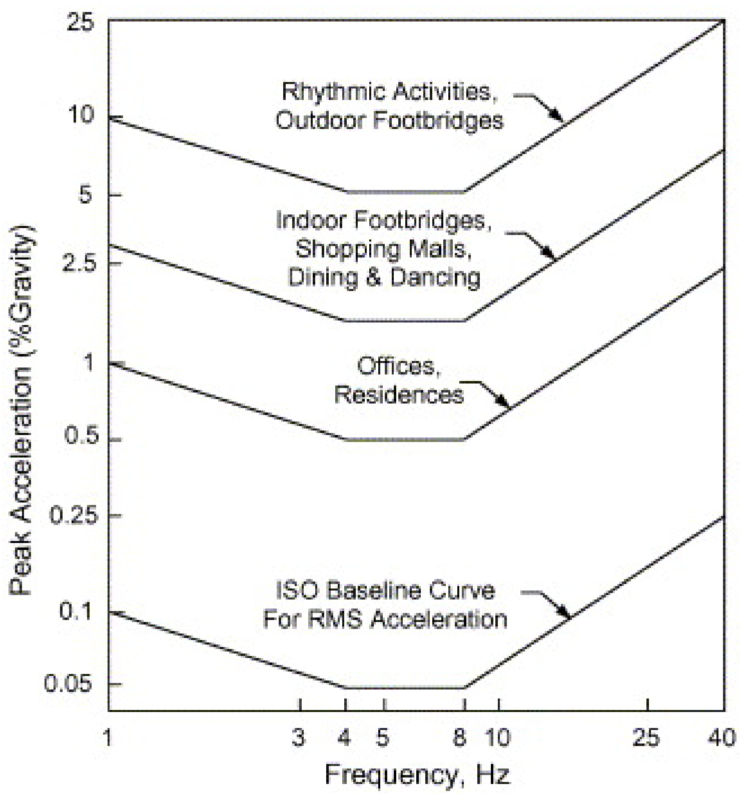

Figure 2-1: ISO Baseline Curve (Ebrahimpour & Sack, 2005)

2.2 Testing

2.2.1 Lenzen

Lenzen tested 46 steel joist-concrete slab floors using a transient excitation, and rated the response of occupants. Classification of the type of excitation was based on observations from the period of vibration decay: if the vibration decayed before 5 cycles, only the initial impact was perceived; if the vibration decayed after 12 cycles, it was perceived as steady-state excitation; otherwise, it was perceived as transient vibration. These conclusions, or associated damping values, were not included in the Modified Reiher-Meister scale. This study was the first to use the heel drop as a means of excitation (Lenzen, 1966).

Factors influencing damping were found to include human occupants. Four occupants increased the damping of a floor by 300% over its unloaded value. Adding dead weight proved to have an opposite result, providing less damping than the unloaded value. Non-structural components, especially partitions, also noticeably increase damping ratio. Ceilings were found to provide a nominal increase in damping, provided they were directly attached to the floor, not suspended (Lenzen, 1966).

Several conclusions were drawn regarding floor response and perception: floor vibrations are transient, and occupant-induced; the main factor influencing human perception is damping; human occupants have a significant influence on the damping of a floor system; and, if vibrations are not sufficiently damped after 12 cycles, the response is equivalent to steady-state, while is the response is sufficiently damped before 5 cycles, the occupant perceived only the initial impact (Lenzen, 1966).

2.2.2 Murray (1979)

In 1979, Murray conducted a review of existing acceptability criteria. Because the existing criteria were inconsistent, and did not incorporate damping, a new criterion was developed based on the results from testing 90 in situ steel and concrete floors subjected to heel drop excitation. The criterion and associated design method is shown in Eq. 2-20.

2.2.3 Onysko

In 1985, an in situ study of 646 residential light-frame wood floor systems was conducted by Onysko in order to assess the performance of the traditional $L/360$ deflection criterion on occupant comfort. At the time of the study, the only serviceability requirement used for design of residential floor systems was the span limit of $L/360$ under specified service loads. Many of the floors designed using the $L/360$ limit proved to be unsatisfactory based on occupant surveys. Results from subjective owner surveys were compared with static deflection and heel drop response measurements in order to develop a satisfactory design criterion (Onysko, 1985).

2.2.4 Kraus and Murray

Kraus and Murray studied 25 residential floor systems supported by cold-formed steel joists in 1997. This is one of the earliest published floor vibration studies which investigated floor supported by cold-formed steel joists. In situ and laboratory floors were tested, as well as two-joist T-beam sections. The objective of the study was to assess the predictive ability of the following design methods: Swedish (Ohlsson, 1988); Australian (AS3623, 1993); American (Johnson, 1994); and, Canadian (Onysko, 1985). The recommended method for use with cold-formed steel joist supported floors was the Canadian method; based on agreement with test results and designer usability (Kraus, 1997).

2.2.5 Waterloo

The Canadian Cold-formed Steel Research Group at the University of Waterloo has conducted studies examining the vibration response of residential floor systems supported with cold-formed steel joists, beginning in 1999. The study by Kraus and Murray was extended to include the Smith and Chui method (Smith & Chui, 1988), Applied Technology Council Design Guide 1 (Allen, Onysko, & Murray, 1999) and Canadian Wood Council (Canadian Wood Council, 1996) design methods (Rizwan, 2000). The accuracy of finite element and ribbed plate models were evaluated, using laboratory test results for comparison (Liu, 2001). Experimental investigation of the influence of ceiling materials, support conditions, subfloor gluing, and blocking, bridging, and screw patterns was conducted (Tangorra, 2005; Xu & Tangorra, 2007). These tests were compared against various design methods, and several in situ floors in low-rise buildings. A refinement to the design methods was proposed by including a semi-rigid connection factor (Tangorra, 2005).

2.3 Acceptability Criteria and Design Methods for Occupant Comfort

2.3.1 Lightweight Floor Systems Supported by Wood Joists

North American residential construction design provisions do not specifically address vibration serviceability. Traditionally, the maximum deflection limit of L/360 for joist span under uniform service loads has been employed. This limit was originally intended to prevent damage to non-structural elements, such as cracking of plaster ceilings. Studies conducted for both residential wood construction (Onysko, 1985), and steel joist-concrete construction (Allen & Murray, 1993) have indicated that the traditional deflection criterion does not satisfy occupant comfort requirements from vibrations due to walking or impact excitations.

Onysko's design static stiffness criterion wasdevised from his testing program. Maximum vertical deflection (y, in mm) from a 1 kN point load applied to the center of the floor is limited by the following expression (Onysko, 1995):

$$y = \frac{8.0}{L^{1.3}} \tag{2-1}$$

where L is the span length of the joists.

For floors with spans less than 3 m, the maximum vertical deflection is limited to 2 mm.

This limit was obtained by correlationing occupant surveys to measured deflection from field testing (Onysko, 1985). A deflection limit is very accessible for design use, but is not based on actual dynamic performance. Onysko's design criterion has been adopted explicitly or in the form of span tables by the National Building Code of Canada (1990). The static deflection limit used in the ATC Design Guide 1 is based on Eq. 2-1 (Allen, Onysko, & Murray, 1999).

Smith and Chui attempted to develop a "designer usable" method for predicting the dynamic properties and response of lightweight, rectangular plan, domestic wooden floors (Smith & Chui, 1988). Their research suggested that no reliable annoyance criteria for domestic floors had been developed. General research in the field of floor vibrations suggested that all systems should be designed such that their fundamental frequency does not fall in the range of 4 Hz to 8 Hz, and that human perceptibility is influenced by amplitude, frequency, and decay components of vibration. Their design method incorporated these aspects by providing a threshold value for the frequency-weighted RMS acceleration response of a floor system to a heel-drop impact.

Their design method is based on a simply-supported, ribbed orthotropic plate model, with properties matching the composite floor system. Fundamental frequency is obtained from this model using the Rayleigh method and the following assumptions:

1. sheathing is thin;
2. orthotropic with axes oriented parallel to the floor axes;
3. joist stiffness is constant throughout the length;
4. bending behavior and vertical displacement occur only;
5. there is no vertical separation between the sheathing and joists under load;
6. all behavior is linear-elastic and small deflection theory is valid; and,
7. sheathing is continuous and without in-plane shear lag effects.

These assumptions are applicable to dimension lumber joists, but cold-formed steel joists have thin webs which will increase the impact of shear deformation, and discontinuities along their length.

Their procedure uses energy methods for estimating the fundamental frequency of the system. A deflected shape is assumed, and the potential strain energy of the system in this position is equated to the kinetic energy of the system at zero deflection. If free vibration and zero damping are assumed, the fundamental frequency of the system can be obtained. This method is relatively easy when compared to the eigenvalue solution required for the Rayleigh-Ritz method. However, additional constraints are introduced by calculating strain energy from fundamental mode deflection only. This adds virtual stiffness to the system and will result in an overestimated fundamental frequency. Another limitation of the Rayleigh Method (and Rayleigh-Ritz Method) is that similar constraints are introduced by assuming the deflected shape, rather than predicting it.

The authors assume that well-built floors have separated natural frequencies, so response will be dominated by the fundamental frequency (f_o), and the following single-degree-of-freedom model can be used:

$$f_o = \frac{\pi}{2a^2}\sqrt{\frac{E_jI_j(n-1)}{\rho_s hb + \rho_j cd(n-1) + 4W_o/a}} \qquad 2\text{-}2$$

where:

- a is the span of the floor;
- b is the width of the floor;
- c is the joist thickness;
- d is the joist depth;
- n is the number of joists;
- ρ_j is the joist density;
- h is the subfloor thickness;
- ρ_s is the subfloor density;
- W_o is the mass of the observer; and,
- E_jI_j is the composite flexural rigidity of a single sheathing and joist t-beam section, calculated as described in the reference.

According to the design procedure, the fundamental frequency must be greater than 8 Hz and the frequency-weighted RMS acceleration response due to heel drop excitation must be checked. The mass of the floor system (m) is expressed as:

$$m = \frac{a}{2}\left[\rho_s hb + \rho_j cd(n-1) + 4W_o/a\right] \qquad 2\text{-}3$$

Acceleration due to a heel-drop, which is assumed to be in impulse function, is calculated using the Duhamel integral; details for which are given in a reference (Chui, 1986). RMS acceleration from heel-drop excitation (A_r) is is approximated using the following expression:

$$A_r = \frac{P_o}{m\omega_n}K(\beta, \omega_n, t_1) \qquad 2\text{-}4$$

where:

P_o is 70% of the weight of the occupant;

m is the mass calculated with Eq. 2-3; and,

ω_n is the angular natural frequency frequency, $2\pi f_o$.

K is an empirically derived function of damping ratio (β), ω_n and heel drop duration (t_1), given by the following formula:

$$K = \sqrt{\frac{\omega_n[1 - e^{-2\beta\omega_n t_1} + (1 - e^{-2\beta\omega_n(1-t_1)})(1 - 2e^{-\beta\omega_n t_1}\cos(\omega_n t_1) + e^{-2\beta\omega_n t_1})]}{4\beta}} \qquad \text{2-5}$$

The authors recommend using values of 2% or 3% for β, 0.05 s to 0.07 s for t_1, and 500 N for P_o. When these assumed values are used, the frequency weighted RMS acceleration, based on the ISO 2631 method of $8/f_o$, is expressed by the following:

$$A_r = \frac{2000K(\beta, \omega_n, t_1)}{m\pi {f_o}^2} \qquad \text{2-6}$$

The dynamic response of the floor system, frequency-weighted RMS acceleration from a heel-drop, is limited to 0.45 m/s^2 in order to satisfy the vibration serviceability requirements of the design procedure.

The frequency and acceleration predictions are based on two key assumptions:

1. the floor is simply supported on all edges; and,
2. the free vibration response is dominated by the fundamental mode.

These two assumptions are not necessarily applicable to high width-to-length floor configurations, so caution should be exercised when applying this method on short, wide floors. The model assumes a well-built floor system to justify the assumption of a dominant fundamental mode, which immediately suggests that the model is not applicable for retrofitting poorly-built floors. It may be reasonable to assume that the outermost effective joists are restrained with a simple support; but the effective width of low aspect ratio floors is most likely a fraction of the total floor width. To evaluate this properly, stiffness in the direction perpendicular to the joists must be considered, and thus more than the fundamental mode will be involved. Unfortunately, the typical residential floor system has a width dimension that exceeds the joist span. The author’s own verification of their method against experimental results shows that it under-predicts the weighted RMS acceleration by up to 20%. The model was designed to be used with dimension-lumber joists, as seen by the simple area and moment of inertia formulas. This portion is not applicable to floors with cold-formed steel joists, but a more applicable revised version of the model for engineered wood joists is given in subsequent publications (Canadian Wood Council, 1996).

A critical response to this paper by Allen and Rainer suggests that while a heel-drop criterion has been used successfully for long-span floors, it may not be applicable to lightweight, short-span, and high frequency wood floors (Allen & Rainer, 1989). The heel-drop is a semi-empirical excitation, and the solution for a response requires the floor to vibrate exclusively at its fundamental frequency. Walking is a more appropriate forcing function, which may result in resonant responses from the floor if its fundamental frequency is lower than 10 Hz. This frequency range is typically not a concern for short-span floors, but Smith and Chui's model suggests that short-span floors have poorer performance in some cases. The response also suggests that discomfort in lightweight floors is generated by deflection due to step forces, not the free-vibration response from an impulse load, suggesting that a point-deflection model may be more appropriate for the specific type of construction examined.

This study also discusses the influence of several construction details, based on observations from experimental results. Decreasing joist spacing was found to increase the primary bending stiffness, and consequently the fundamental frequency, of the floors. However, this may not provide a better-performing floor because it was also found to reduce the spacing between vibration modes, which can increase the floor response and invalidate the assumptions used in the design model. Supporting all sides of the floor, instead of just the ends, was found to increase the damping ratio and slightly increase the fundamental frequency. The greatest impact of supporting all sides of the floor was an increased spacing between natural frequencies, thus, increased modal separation. Perpendicular stiffening via blocking between the joists was also found to increase modal separation.

2.3.2 Combination of Static and Dynamic Criteria (Ohlsson, 1988)

A design criterion for lightweight floor systems, independent of construction, was developed by Ohlsson (Ohlsson, 1988). It is applicable to floor systems with fundamental frequencies great than 8 Hz, avoiding the sensitive frequencies for the human body. The following criteria must be met for the floor to be acceptable:

1. deflection due to a 1 kN point load must be less than 1.5mm;
2. impulse response and damping must be in an acceptable region; and,
3. root-mean-square (RMS) acceleration from steady state excitation must be lower than an appropriate limiting value.

The impulse used in the testing was a heel drop. Ohlsson provided a plot of descriptive ranges based on the impulse velocity and damping coefficient (σ_o), the product of frequency and damping ratio, which is shown in Figure 2-2. No limiting value is provided for RMS acceleration, the suggested procedure is to use the value on a similar floor which is built and acceptable to the occupants, which makes this portion of the criteria open to misinterpretation (Ohlsson, 1988).

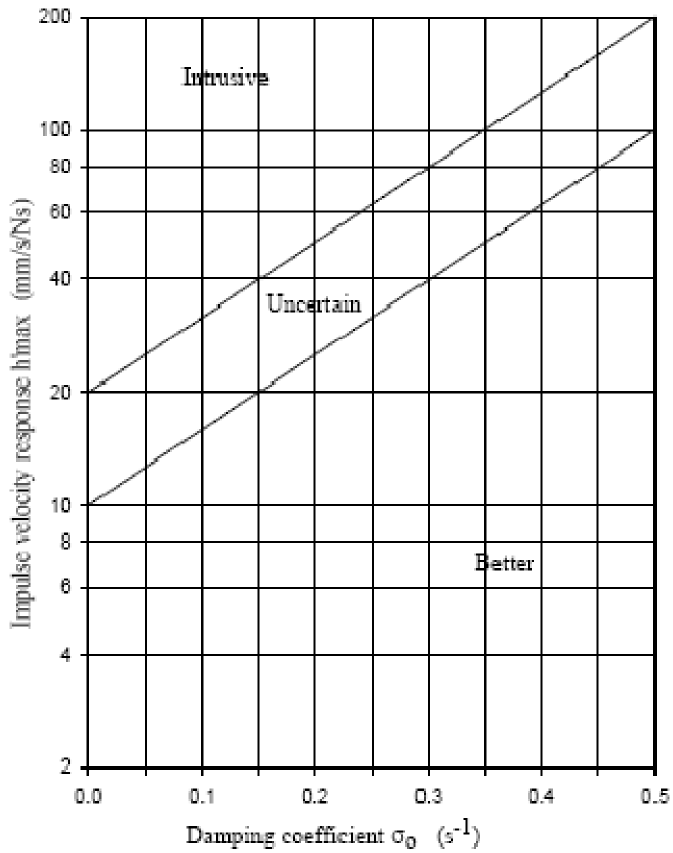

Figure 2-2: Ohlsson's Classification of Floor Response to an Impulse Load (Kraus, 1997)

The Australian Standard Domestic Metal Framing Code (1993) contains a vibration serviceability design procedure based on Ohlsson's criteria. The procedure is very similar to the Swedish Council for Building Research (Ohlsson, 1988) design procedure, with the inclusion of effective floor width for the mid-span deflection calculation. For this reason, the Australian procedure is used for the following discussion.

Ohlsson's criteria can only be applied to floors with a fundamental frequency in excess of 8 Hz. The fundamental frequency *(f_o)* is estimated using the following:

$$f_n = \frac{\pi}{2}\sqrt{\frac{D_x}{gL^4}}\sqrt{1+\left[2n^2\left(\frac{L}{B}\right)^2 + n^4\left(\frac{L}{B}\right)^4\right]\frac{D_y}{D_x}} \quad \text{2-7}$$

where:

D_x is the flexural stiffness parallel to the joists (EI_t/S);

D_y is the flexural stiffness perpendicular to the joists ($Et^3/12$);

S is the joist spacing;

t is the subfloor thickness;

L is the floor span;

n is the modal number (1 for fundamental frequency); and,

B is the floor width.

A live load of 0.3 kPa is recommended as an estimated service load. In order to apply Ohlsson's criteria, the estimated fundamental frequency must be greater than 8 Hz.

For the first criterion, the maximum static deflection due to a concentrated load of 1 kN is limited to 2 mm. Mid-span deflection of a single joist (δ) is estimated with simple beam behavior using the following equation:

$$\delta = \frac{P_d L^3}{48EI_t} \quad \text{2-8}$$

where:

P_d is the effective applied load calculated by $k_d P$; and,

P is 1 kN.

The effective width factor (k_d) is calculated by the following:

$$k_d = 0.833 - 0.34\log_{10}\left[\left(\frac{k_c}{k_b}\right) + 0.44\right] \quad \text{2-9}$$

$$k_c = \frac{D_y L}{s^3} \quad \text{2-10}$$

$$k_b = \frac{EI_t}{L^3} \quad \text{2-11}$$

For the second criterion, the estimated maximum velocity response (V_{max}) from a 1 Ns impulse is determined by the following expression:

$$V_{max} = 4\left(\frac{0.4 + 0.6N_{40}}{wLB + 200}\right) \quad \text{2-12}$$

where:

w is floor mass, including service load (kg/m^2);

L is floor span (m); and,

B is floor width (m).

N_{40} is calculated by the following:

$$N_{40} = \frac{B}{L}\sqrt{\sqrt{\frac{r + \frac{40}{f_o} - 1}{r}} - 1} \quad \text{2-13}$$

where:

r is ratio of composite bending stiffness in the joist and transverse directions.

Once calculated, the estimated maximum velocity response is limited by the following:

$$\log_{10}(V_{max}) < 1.2 + 2\sigma_o \quad \text{2-14}$$

where:

σ_o is the damping coefficient ($f_o\zeta$); and,

ζ is the modal damping ratio (assumed to be 0.9%).

The RMS acceleration requirement from Ohlsson's criteria is not included in this procedure because an explicit limit is not given.

Johnson developed a design procedure for timber floors from a study which originally was intended to be based on Murray's (Murray, 1991) criterion (Johnson, 1994). The criterion used for this design procedure was simplified to limiting the fundamental frequency of a floor (supporting only self-weight) to a minimum value of 15 Hz. The natural frequency of the floor system (f_{system}) is computed using Dunkerley's expression, which is a method of combining fundamental frequencies of the joist (f_{joist}) and girder (f_{girder}) panel modes of the floor system. It is shown in the following equation:

$$f_{system} = \sqrt{\frac{f_{joist}^2 \cdot f_{girder}^2}{f_{joist}^2 + f_{girder}^2}} \quad \text{2-15}$$

The method provides Eq. 2-16 for estimating the fundamental frequency of the joists, ignoring composite action with the subfloor:

$$f = 1.57\sqrt{\frac{gEI}{Wl^3}} \qquad \text{2-16}$$

where:

g is acceleration due to gravity (386.4 in/s^2);

EI is the combined elastic modulus and moment of inertia of a joist;

W is the dead weight of the floor system (lbs); and,

l is the effective length of joists (in).

The effect of sheathing was found to be negligible for the joist panel frequency calculation was based on tests involving wood floors, where the sheathing stiffness is generally negligible in comparison to the joist stiffness. The 15 Hz criterion applies only to floors supporting their own weight, and is intended to be used as a design tool for determining whether the floor will be acceptable for vibration due to footfalls (Kraus, 1997).

2.3.3 Heavy Concrete Slab Floor Systems Supported by Hot-Rolled Steel Joists

Reiher and Meister studied the subjective reactions of individuals to five minute periods of steady-state floor vibrations. Their results were published as a scale of degrees of acceptability based on the frequency of vibration and peak displacement. This scale, know as the Reiher-Meister scale, was the first design criterion developed specifically for floor vibration serviceability (Reiher & Meister, 1931).

Lenzen recognized that typical floor vibrations are the result of transient excitations. Subjective occupant evaluations for several different steel-joist concrete floors excited with heel drops were obtained. These results were found to correlate with the Reiher-Meister scale, multiplied by a factor of 10; to account for the decay of vibration (Lenzen, 1966). The resulting vibration serviceability criterion is known as the Modified Reiher-Meister scale, shown in Figure 2-3; with fundamental frequency of the floor plotted against the maximum amplitude from a heel drop, which is a calculated value based on a composite T-beam section.

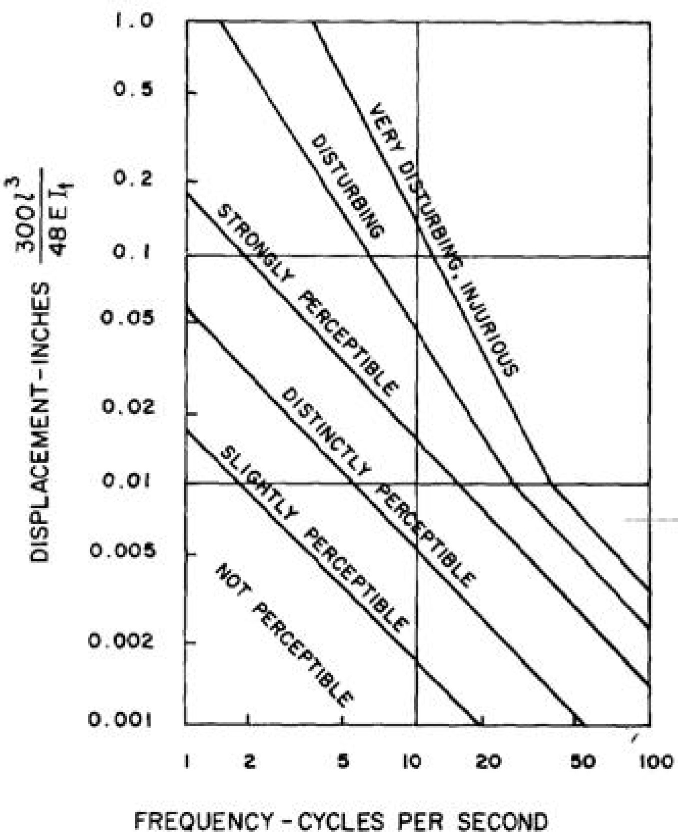

Figure 2-3: Modified Reiher-Meister Scale (Lenzen, 1966)

Lenzen found that the fundamental frequency (f) of the floors was governed by one-way behavior of the composite section, and could be predicted with the following:

$$f = 1.57\sqrt{\frac{gEI_t}{w_d l^4}} \qquad \text{2-17}$$

where:

g is acceleration due to gravity (386.4 in/s^2);

EI_t is the combined elastic modulus and moment of inertia of the composite section (multiplied by the number of joists);

w_d is the dead weight of the floor system (lbs/in); and,

l is the effective length of joists (in).

This model does not specifically incorporate the transverse properties of the floor system, and assumes that all joists are equally participating in the dynamic response. The paper does suggest that 12 be the maximum number of joists used, and recommends the use of 10, but no basis is given for these assumptions (Lenzen, 1966). The predicted fundamental frequency is dependent on this assumed number of joists.

The values of natural frequency and deflection under a 1334 N (300 lb) dynamic load are used to obtain a qualitative estimate of the floor's response. The deflection (Δ), in inches, is found with the following formula:

$$\Delta = \frac{300l^3}{48EI_t} \tag{2-18}$$

Lenzen's recommended design procedure is to calculate the deflection and frequency of a floor system and ensure the values correspond to an acceptable position on the Modified Reiher-Meister Scale (Figure 2-3). Because this method does not directly consider damping and transverse stiffness, it is only applicable to floor systems similar to the ones tested by Lenzen: concrete slabs supported by hot-rolled beams.

An empirical design criterion based on the response of the floor to a calculated heel drop excitation was developed by Allen and Rainer (Allen & Rainer, 1989). It is generally acceptable for use with steel joist-concrete slab floors with fundamental frequencies below 10 Hz and spans greater than 7.62 m (25 ft). The initial acceleration due to a heel drop a_o is estimated by the following:

$$a_o = 0.9\frac{2\pi fI}{M} \tag{2-19}$$

where:

f is the fundamental frequency of the floor (Hz);

I is a 15 lb-s impulse; and,

M is the effective vibrating mass (lb).

The width of the floor panel, for a concrete slab floor, is estimated as 60 times the slab thickness. The estimated acceleration is then compared against the limits shown in Figure 2-4. The limit is for walking excitation, but based on heel-drop testing.

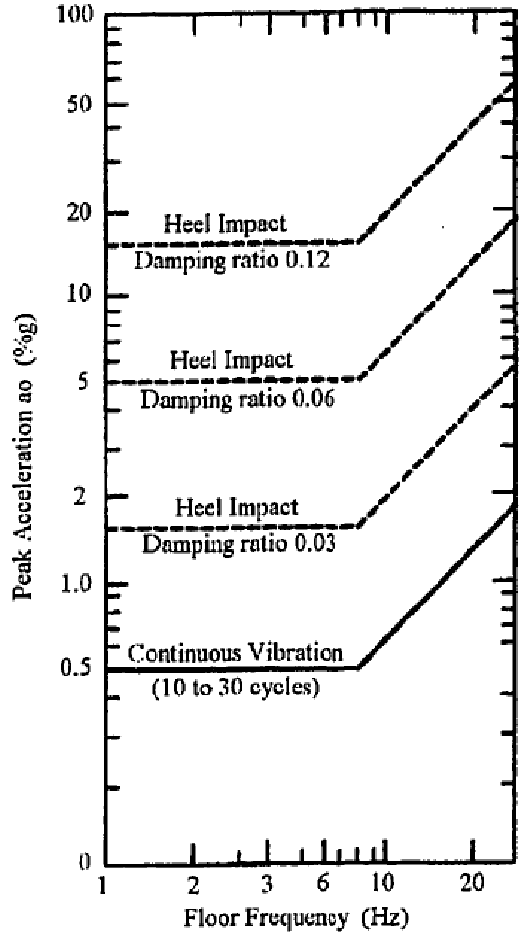

Figure 2-4: Acceptability Limit of Floor Vibration due to Walking by Allen and Rainer (Xin, 1996)

The damping values in Figure 2-4 are based on suggested for different types of floors: 0.03 for bare floors; 0.06 for finished floors (with ceiling, ducts, flooring, furniture); and, 0.12 for finished floors with partitions. This method has been included as an appendix in outdated editions of the CSA S16 Code.

In recognizing that office and residential vibrations are generated by transient excitations, where damping plays a critical role in perception, Murray proposed the design criterion for a minimum damping ratio, given by the following limit:

$$D > 35A_of + 2.5 \quad \text{2-20}$$

where:

D is the damping ratio (% of critical);

A_o is the initial amplitude due to a heel drop (in), using an estimated number joists; and,

f is fundamental frequency, using a composite T-beam model.

In order to calculate the acceleration response, the actual heel drop approximated by a ramp function decreasing linearly from 2669 N (600 lb) to 0 over 50 milliseconds. This method is not recommended for floors with fundamental frequencies greater than 10 Hz (Murray, 1991). Because lightweight floor systems typically have fundamental frequencies greater than 10 Hz, it is not applicable to floor supported by wood or cold formed steel joists in most cases.

A design criterion was developed by Allen, Rainer and Pernica (Allen, Rainer, & Pernica, 1985) to be used for the design of floors exposed to rhythmic excitations, such as aerobics. Rhythmic excitations are periodic in nature, and differ from walking excitation, which can be assumed to be several repeating transient impacts. The design procedure by Allen and Murray (1993) is an extension of this work, applied specifically to walking excitation.

This design procedure recognizes that several different occupancies can be affected by excitation at one location in a building, and recommends designing to an acceleration limit based on the most sensitive occupancy present. The acceleration limits, shown in Table 2-1, are based on the ISO criterion and the authors' experience.

Table 2-1: Recommended Acceleration Limit due to Rhythmic Activities (Allen, Rainer, & Pernica, 1985)

Critical Occupancy	Acceleration Limit (%g)
Office and residential	0.4 – 0.7
Dining and weightlifting	1.5 – 2.5
Rhythmic activity only	4 - 7

A design criterion and procedure for floors excited by walking vibration was proposed by Allen and Murray (Allen & Murray, 1993). The procedure is intended to be used for steel joist-supported floors with fundamental frequencies below 9 Hz, because it assumes that the response due to walking is resonant. It can be extended to other floors, but may be less accurate.

The design criterion is based on a limiting value for acceleration response, which is based on the ISO baseline curve and associated multipliers for intended occupancy. The response of the floor is based on a modal summation of the response to individual harmonics from walking excitation. The harmonics are defined as multiples of the frequency of the footfalls, or step frequency. The forcing function was determined to be of the following form:

$$F(t) = P\left[1 + \sum_{i=1}^{\infty} \alpha_i \cos(2\pi i f_{step} t)\right] \quad \text{2-21}$$

where:

P is the weight of the walking person;

α_i is the dynamic load factor of the ith harmonic;

i is the harmonic multiple number; and,

if_{step} is the step frequency of the ith harmonic (Hz).

Suggested values, based on testing, for the dynamic load factor and step frequency are given in Table 2-2.

Table 2-2: Walking Forcing Function Components

Harmonic **(i)**	**Step Frequency Range** **(if_{step})**	**Dynamic Load Factor** **(α_i)**
1	1.6 – 2.2	.5
2	3.2 – 4.4	.2
3	4.8 – 6.6	.1
4	6.4 – 8.8	.05

The response of the floor is predicted using a simplified SDoF beam model vibrating with the forcing function applied at mid-span. For each harmonic component of the applied force, the resonant acceleration response $\left(\frac{a_i(t)}{g}\right)$ is given by:

$$\frac{a_i(t)}{g} = \frac{\alpha_i P}{0.5W}\frac{R}{2\beta}\cos(2\pi i f_{step} t) \quad \text{2-22}$$

where:

W is the assumed beam weight;

β is the damping ratio; and,

R is a reduction factor (0.5 for two-way floor systems).

Peak acceleration, for comparison to the ISO criterion, can be determined by taking the maximum value of one for the cosine expression. The fundamental frequency of the oscillating beam is implicitly included in Eq. 2-22 as $\frac{0.5W}{g}$.

The recommended design procedure involves selecting a maximum acceleration for the floor system based on the ISO requirements and inverting the expression for peak acceleration in order to determined the required combination of floor fundamental frequency and βW so that the acceleration limit is not exceeded at resonance. When the dynamic load factor is approximated by a log-linear curve, this design criterion becomes the following:

$$\frac{a_o}{g} = \frac{Pe^{(-0.35f)}}{\beta W} \qquad \text{2-23}$$

The previously discussed acceptability criteria have proven suitable for design of floor systems that generally agree with the types of construction, occupancy, and loading in which they were developed. The evolution of floor testing and design from Reiher and Meister, where amplitude and frequency of response were the only factors considered, to transient excitations, and the incorporation of damping, walking excitation, and resonant response indicates that research is moving towards a more specific understanding of the floor vibration phenomenon. There is still a continuing need for research in the field of cold-formed steel floor vibration in order to provide accurate models of the specific behavior, and assist designers in the creation of satisfactory floor systems.

Chapter 3. Experimental Investigation

The experimental investigation presented in this thesis encompasses a laboratory investigation conducted at the University of Waterloo Structures Laboratory, and a field investigation conducted at four locations in the United States. The laboratory investigation involved 23 laboratory test floor configurations; each a unique configuration based on variations in span, joist cross-section, subfloor assembly, and construction details. The two major objectives of the laboratory investigation were:

1. to determine the influence of construction details on the modal properties and dynamic response of the floor systems; and,
2. to assess the accuracy of several published vibration models by validating them with the modal properties and dynamic response of the laboratory test floors.

Discussion of the first objective became the subject of another body of work, which also contains a detailed discussion of all the testing apparatus, procedures which were used for the experimental investigation (Davis, 2008). The discussion of the influence of construction detals on the damping ratio and transverse stiffness of the floor systems is expanded upon in this thesis, and the vibration model assessment is presented.

The field investigation involved 12 in situ test floors in residential mid-rise buildings, and 8 in situ test floors in purpose-built sound test chambers. The two major objectives of the field investigation were:

1. comparing the performance of laboratory and in situ floors of similar configurations; and,
2. assessing the dynamic response of the in situ floors in terms of relevant vibration serviceability criteria and determining which floors may have occupant comfort problems.

In similar fashion to the laboratory investigation, these two objectives were briefly discussed in a separate study (Davis, 2008). They will be expanded upon in this thesis.

This Chapter will discuss the technical aspects and procedures of the laboratory and field investigations, and the data processing methods used. The naming convention for the test floors is introduced, and all relevant construction details of each floor are presented. An expanded discussion on the test procedures, laboratory apparatus, data acquisition system and data processing techniques can be found in Davis (2008).

3.1 Laboratory Investigation

3.1.1 Test Floor Support Apparatus

All laboratory floor systems were built in place and supported by a large hot-rolled steel test frame constructed of 305 mm (12 in) w-sections. The test frame supported the floor at both ends, and was adjustable so that spans of up to 7.32 m (24 ft) could be accommodated. The maximum floor width accepted by the test frame was 4.88 m (16 ft). The maximum number of joists based on the 610 mm (2 ft) spacing was nine, and all floors were constructed with nine joists. A sketch of the test frame is shown in Figure 3-1. In an effort to ensure the test frame was sufficiently stiff and massive to dynamically isolate the test floor, the center support columns were filled with concrete.

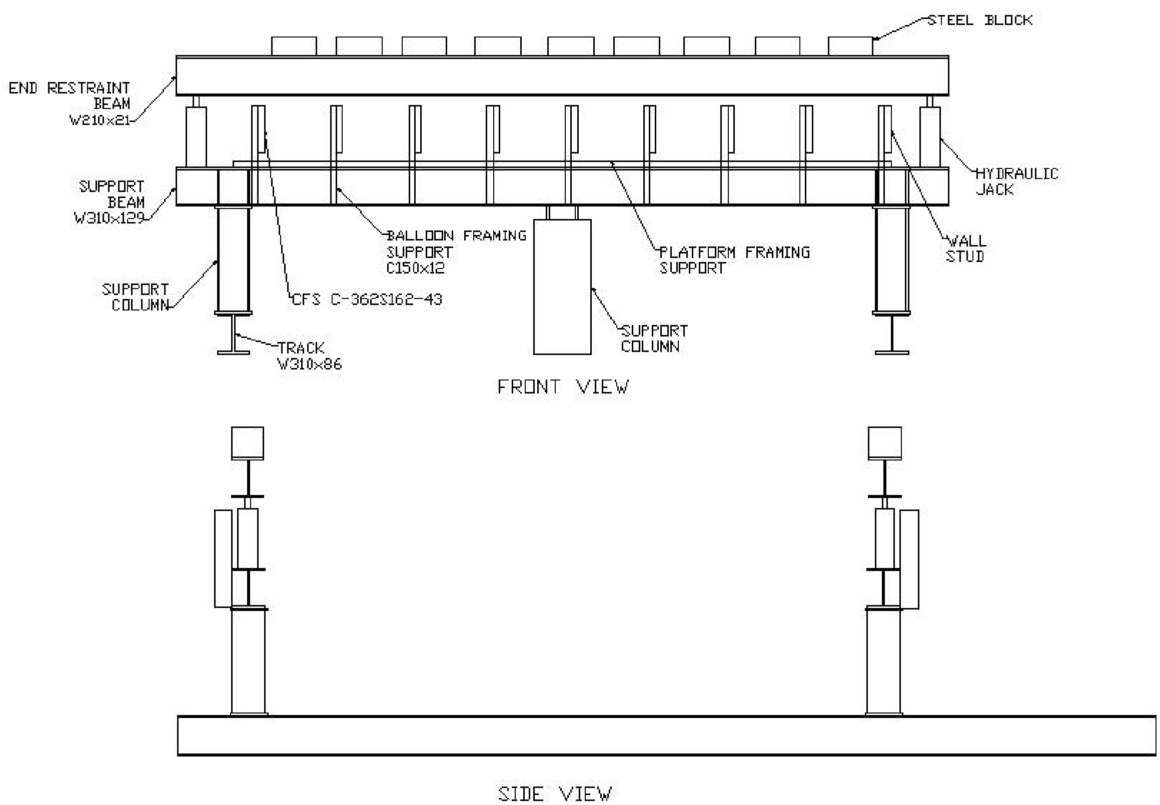

Figure 3-1: Laboratory Test Frame (Davis, 2008)

The test frame supported the laboratory test floors at the ends, and the edge joists were left unsupported. The resulting support condition was supported on two sides and free on the other two, which allowed for simple modeling, and provided conservative results when compared all sides being supported (Davis, 2008). Three end-framing conditions were approximately provided by the supports: balloon framing, platform framing with bearing-induced rotational restraint, and platform framing without rotational restraint.

Balloon framing is a technique where the floor system and wall studs are not "stacked" like traditional platform framing, or stick framing. The floor joists span only the framed space between the wall studs, or supporting girders. The entire floor is connected through a shear connection to the gravity load-bearing frame of the building. In platform framing, the floor joist span exceeds the framed opening, and the floor is supported on wall studs or girders through a bearing connection. The gravity load-bearing structure above the floor system rests on top of the bearing portion of the floor joists in platform framing, applying a certain degree of end-fixity. Figure 3-2 illustrates typical balloon and platform framing in an in-line framed structural system.

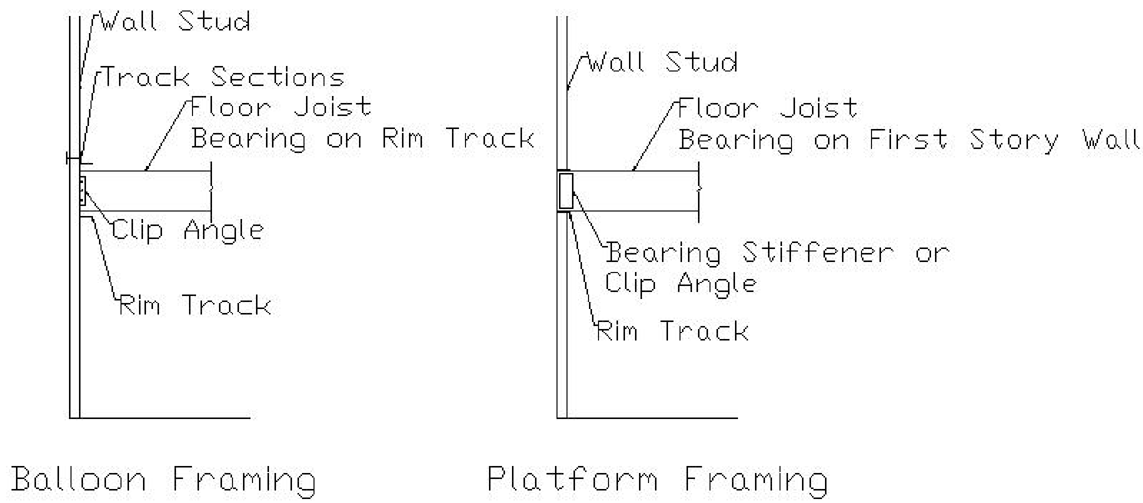

Figure 3-2: Balloon and Platform End-Framing Conditions (Davis, 2008)

For the purpose of this thesis, the platform framing end-support condition with bearing-induced rotational restraint will be referred to as platform framing. The presence of an upper-story bearing on the floor system is typical in mid-rise structures. The platform framing end-support condition without rotational restraint will be referred to as simple support in order to maintain distinction between conditions, and because it is the best approximation of the "pin-pin" case of the floor systems tested.

Balloon framing was achieved by fastening the rim track to sections of wall stud that were spaced in-line with the floor joists. The wall stud sections were secured to the balloon framing supports with several screws through the web of the studs. For the platform and simple support conditions the floor joists were bearing on top of a standard 2 x 4 section of dimension lumber, spanning the entire floor width, which was bolted to the test frame. For the platform framing case, a bearing line load of 1.9 kN/m (130 lb/ft) was applied above the bearing support with a weighted end-restraint beam. For the simple support case, the beam was raised off the floor with hydraulic jacks. Specific details for the laboratory end-framing conditions are given in Davis (2008).

3.1.2 Construction Details and Materials

All laboratory test floors were built with nine cold-formed steel joists spaced at 610 mm (24 in) on center. The joist webs were connected to punched shear tabs on a proprietary rim track at the ends of the floor. Every floor had a row of blocking and strapping installed at 2.44 m (8 ft) on center; one row for floors 4.88 m (16 ft) and shorter and 2 rows for floors up to 7.32m (24 ft). Specific material and fastener specifications and patterns are given in greater detail in Davis (2008).

Two types of cold-formed steel C-shape joists were used:

1. Dietrich TradeReady® Steel Joists, 305 mm (12 in) deep, with section properties defined by the TDW product code: which indicates a 50.8 mm (2 ft) flange (abbreviated with TDW); and,
2. Dietrich C Stud joists, 305 mm (12 in) deep, with section properties defined by the CSW product code: which indicates a 50.8 mm (2 ft) flange (abbreviated with CSW).

All cold-formed steel joists contain small web openings so that other structural components can pass through them. The TDW joists are a proprietary design which incorporating large lip-reinforced circular openings so that ducts and other services can be passed trough the joists, eliminating the need for a drop ceiling or ill-advised cuts made into the joists by tradespeople. The openings in the TDW joists are 203 mm (8 in) in diameter, and spaced at 1.22 m (4 ft) intervals. The largest openings in the CSW joists are 101.6 mm x 38.1 mm (4 in x 1.5 in) elliptical openings, without lip reinforcement, spaced at 1.22 m (4 ft) intervals. Joists were installed singly or back-to-back, which is denoted by a (2) in front of the joist type.

There were four subfloor configurations tested: two with a lightweight concrete topping, and two with bare subfloor sheathing. The subfloor configurations were:

1. 19 mm (0.75 in) oriented-strand board (OSB) with tongue and groove joints;
2. 19 mm (0.75 in) FORTACRETE® Structural Panel with tongue and groove joints;
3. 19 mm (0.75 in) FORTACRETE® Structural Panel with tongue and groove joints, topped with a 19 mm (0.75 in) lift of LEVELROCK® Floor Underlayment; and,
4. 0.75 mm (0.0295 in) UFS cold-formed steel form deck with flutes in the transverse direction, topped with a 38.1 mm (1.5 in) lift (maximum depth) of LEVELROCK® Floor Underlayment.

FORTACRETE® Structural Panels are fiber-reinforced cement panels that are non-combustible, and are designed to be installed in a similar fashion to OSB sheets, with the exception that a bead of construction glue is no put on the top flange of the joists for FORTACRETE®. FORTACRETE® Structural Panels are intended for use in both topped and bare applications. LEVELROCK® Floor Underlayment is a gypsum-based, self-leveling floor topping which is intended for use in applications were a lightweight concrete is required. The UFS cold-formed steel form deck had a 14.3 mm (0.56 in) depth and 63.5 mm (2.5 in) pitch.

In subsequent sections: FORTACRETE® Structural Panels will be referred to as FORTACRETE (abbreviated as FC); LEVELROCK® Floor Underlayment will be referred to as LEVELROCK (abbreviated as LR); and, UFS cold-formed steel form deck will be referred to as cold-formed steel deck (abbreviated as UFS).

Several floor configurations had a 15.9 mm (0.625 in) gypsum board ceiling installed on the underside of the joists. The gypsum board was screwed to lengths of resilient channel, which ran perpendicular to the joist direction at 0.305 m (1.0 ft) intervals. The resilient channel was screwed to the bottom flange of the joists at each intersection, and the ceiling was screwed to the resilient channel exclusively. Type X gypsum board was used for floors which were 4.42 m (14.5 ft) long, and Type C gypsum board was used for all longer floors.

A C-shape stiffening member, oriented perpendicular to the joist direction, was added in some configurations. The common name of this detail is a strongback. The strongback passes through the openings in the joists at mid-span. Clip angle brackets are screwed in place to connect the web of the strongback and web of the joists at each joist intersection. The Dietrich CSJ Stud, 92.1 mm (3.625 in) deep with a thickness of 1.44 mm (0.0566 in), was used for the strongback. The two configurations of strongback included in the construction details were:

1. free, where the ends of the strongback were not restrained from vertical displacement as they extended past the outer joists; and,
2. restrained, where the ends of the strongback were restrained from vertical displacement by a clip angle connecting the web of the strongback to a large hot hot-rolled pedestal, as it extended past the outer joists.

A typical plan layout and cross section of a laboratory test floor is presented in Figure 3-3. Note that the blocking pattern is asymmetric due to the odd number of joists tested. This offset was consistently replicated for all laboratory test floor configurations. Additionally, there were no web stiffeners at the platform framing bearing locations. The punched shear tab of the proprietary rim track provided sufficient web stiffening to prevent joist web crippling under the test loads. Detailed construction drawings of most laboratory test floor configurations are included in Appendix A of Davis (2008).

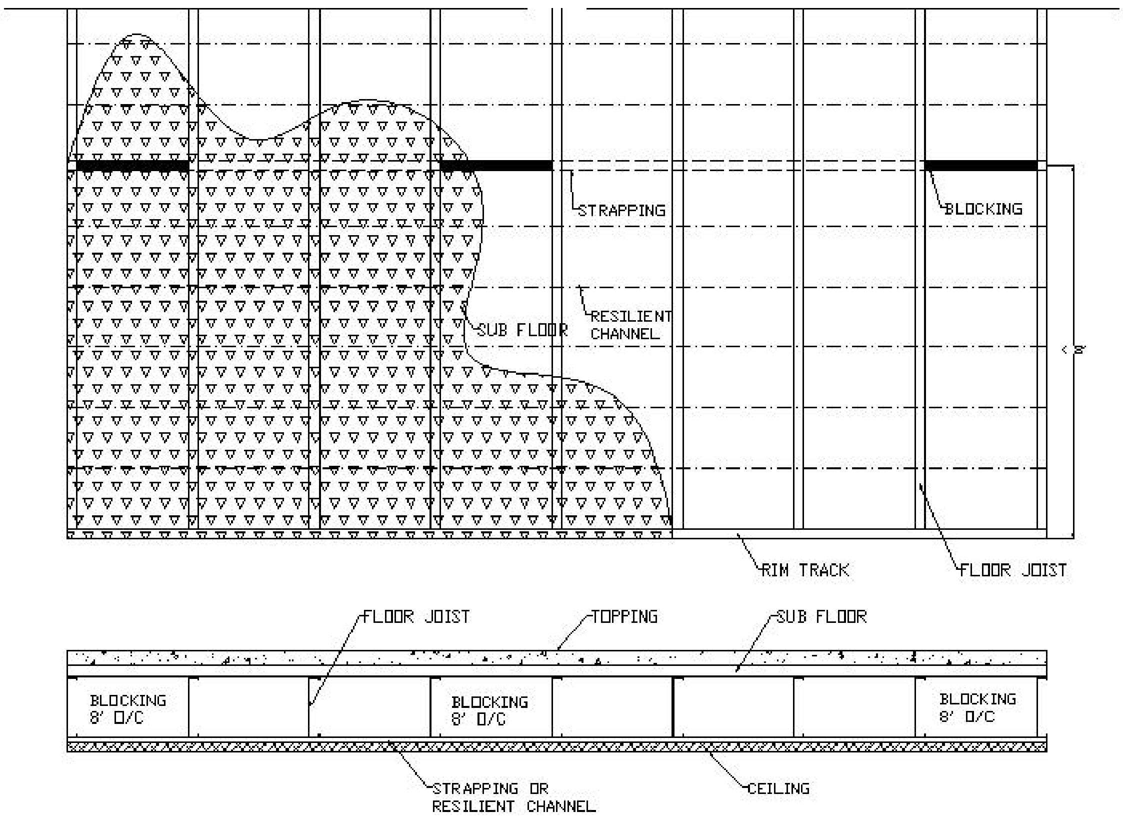

Figure 3-3: Typical Laboratory Test Floor Diagram (Davis, 2008)

3.1.3 Laboratory Test Floor Configuration Descriptions

All laboratory test floor configurations are listed in Table 3-1. The nomenclature contains “LF” indicating that the floors were built and tested in the laboratory. The name also contains a number which indicates the length of the floor, in feet; followed by a letter which distinguishes the configuration. A subscript Roman numeral is used to indicate a partial configuration, where all tests were not conducted for all end-framing conditions.

Table 3-1: Laboratory Test Floor Configurations

Floor Name	Floor Span (ft)	Joist	Joist Depth (in)	Joist Gauge	Subfloor	Ceiling	Strongback
LF14.5A	14.5	CSW	12	16	OSB	-	-
LF14.5A_i	14.5	CSW	12	16	OSB	-	-
LF14.5B	14.5	CSW	12	16	FC	-	-
LF14.5B_i	14.5	CSW	12	16	FC	-	-
LF14.5C	14.5	TDW	12	16	FC	-	-
LF14.5D	14.5	TDW	12	16	FC	Type X	-
LF14.5D_i	14.5	TDW	12	16	FC	-	-
LF14.5E	14.5	TDW	12	16	FC with 0.75 in LR	Type X	-
LF14.5F	14.5	TDW	12	16	UFS with 1.5 in LR	Type X	-
LF17.0A	17.0	TDW	12	14	FC with 0.75 in LR	Type C	-
LF17.0B	17.0	TDW	12	14	FC with 0.75 in LR	Type C	Free
LF17.0C	17.0	TDW	12	14	UFS with 1.5 in LR	Type C	-
LF17.0D	17.0	TDW	12	14	UFS with 1.5 in LR	Type C	Free
LF19.5A	19.5	TDW	12	14	FC with 0.75 in LR	Type C	-
LF19.5A_i	19.5	TDW	12	14	FC with 0.75 in LR	-	-
LF19.5A_{ii}	19.5	TDW	12	14	FC with 0.75 in LR	-	Restrained
LF19.5A_{iii}	19.5	TDW	12	14	FC with 0.75 in LR	Type C	Restrained
LF19.5A_{iv}	19.5	TDW	12	14	FC with 0.75 in LR	Type C	-
LF19.5B	19.5	TDW	12	14	UFS with 1.5 in LR	Type C	-
LF19.5B_i	19.5	TDW	12	14	UFS with 1.5 in LR	-	-
LF19.5B_{ii}	19.5	TDW	12	14	UFS with 1.5 in LR	-	Restrained
LF19.5B_{iii}	19.5	TDW	12	14	UFS with 1.5 in LR	Type C	Restrained
LF19.5B_{iv}	19.5	TDW	12	14	UFS with 1.5 in LR	Type C	-
LF21.8A	21.8	(2)TDW	12	16	UFS with 1.5 in LR	Type C	-

1 ft = 0.3048 mm
1 in = 25.4 mm

3.2 Field Investigation

3.2.1 Test Floor Support Apparatus

The field test floors were tested at for multi-unit residential buildings where the floors were constructed. The floors were supported by the structural system of the buildings in which they were located. The floor systems tested in the field investigation portion of this study fall into two categories: in situ floors, and sound chamber floors. The in situ floors were located in mid-rise residential buildings which consisted of typical condominium units, and employed repetitive floor plan. Floors located at the following three developments were tested:

1. Carlyle's Watch in Columbus, Ohio, an eight story building (abbreviated as CW);
2. Ocean Keyes in North Myrtle Beach, South Carolina, a four story building (abbreviated as OK); and,
3. City Green in Milwaukee, Wisconsin, an eight story building with a podium shared between three towers (abbreviated as CG).

The sound chamber floors were located in purpose-built two story test buildings at the Dietrich Design Group office in Hammond, Indiana (abbreviated as DDG). There were eight identical buildings, each housing a unique floor system. Four of the DDG floor systems (DDG5 to DDG8) were built with structural systems that were not based on cold-formed steel joist framing, and are not discussed in this thesis. Floor plans for the DDG test floors are provided in Appendix A of this thesis. All other floor plans and construction details can be found in Appendices A and B of Davis (2008).

3.2.2 Construction Details and Materials

Unlike the laboratory test floors, it was not possible to maintain uniformity in the construction layout of the field test floors. A limited selection of test sites resulted in floors where the width and number of joists present varied significantly. Other details which differed from site to site included the layout of the services and fire protection in the inter-story depth of the floor, and ceiling construction method. Full-height partitions, such as walls, and partial height partitions, such as kitchen islands, were present on some of the in situ test floors.

All in situ floors were tested at the same stage of completion. The LEVELROCK was in place and fully cured, and there was a ceiling in place. With the exception of one model suite, there were no finishes installed on the floor surface, and the floors were unloaded. All in situ floors were constructed with Dietrich TradeReady® Steel Joists, 305 mm (12 in) deep with section properties defined by the TDW product code, which indicates a 50.8 mm (2 in) flange. The joists were spaced at 610 mm (24 in) on center, and connected to a proprietary rim track at the ends of the floor, fastened with screws between the webs of the joists and the punched shear tab of the rim track. Every floor had a row of blocking and strapping installed at 2.44 m (8 ft) on center; one row for floors 4.88 m (16 ft) and shorter and 2 rows for floors up to 7.32m (24 ft). Layout and connection details were based on the specifications provided to the contractor by the joist manufacturer.

There were three subfloor configurations tested, two with a lightweight concrete topping. The subfloor configurations were:

1. 19 mm (0.75 in) FORTACRETE® Structural Panel with tongue and groove joints;

2. 19 mm (0.75 in) FORTACRETE® Structural Panel with tongue and groove joints, topped with 19 mm (0.75 in) lift of LEVELROCK® Floor Underlayment; and,
3. 0.75 mm (0.0295 in) UFS cold-formed steel form deck with fluted oriented perpendicular to the joist direction, topped with a 38.1 mm (1.5 in) lift (maximum depth) of LEVELROCK® Floor Underlayment.

The topped FORTACRETE subfloor assembly was used in the City Green development. This assembly also included a mineral fiber sound reduction board between the FORTACRETE panels and LEVELROCK topping. This was not the case for the laboratory and DDG test floors with FORTACRETE subfloor assemblies. The acoustical underlayment used was 9.5 mm (0.375 in) USG LEVELROCK® Sound Reduction Board (abbreviated as SRB), with a density of 384.4 kg/m^3 (24 lb/ft^3). The SRB was also employed in the subfloor assembly for DDG3. It was placed at a depth of 25.4 mm (1.00 in) below the surface of the topping, which was set in 2 lifts in order to bond to the UFS.

All field test floors had a 15.9 mm (0.625 in) gypsum board ceiling installed on the underside of the joists. The ceiling was connected to a resilient channel grid that was suspended from the bottom of the joists, at the Carlyle's Watch and Ocean Keyes developments. The ceiling was connected to a resilient channel grid that was fastened directly to the bottom flange of the joists, at the City Green development and DDG site. Double layers of gypsum board were used on some of the DDG floors, indicated with (2). Type C gypsum board was used for all floors.

The strongback detail was not present in any of the field floor systems tested. Davis (2008) contains detailed descriptions of the individual in situ test floors in Section 3.4, and construction drawings of most of the in situ test floors, in Appendix B. A list of the field test floors and relevant unique details is given in Table 3-2.

Table 3-2: Field Test Floor Configurations

Name	Joist Type	Floor Span (ft)	Floor Width (ft)	Subfloor	Ceiling
CG601	TDW	17.5	13.8	FC with LR and SRB	RC
CG604	TDW	14.8	16.9	FC with LR and SRB	RC
CG805	(2)TDW	21.2	28.0	FC with LR and SRB	RC
CGMH6	TDW	16.8	23.8	FC with LR and SRB	RC
CGMH7	TDW	16.8	23.8	FC with LR and SRB	RC
CW707	TDW	16.8	25.8	UFS with LR	Drop
CW708	TDW	14.5	28.5	UFS with LR	Drop
CW709	(2)TDW	21.8	26.3	UFS with LR	Drop
CW805	TDW	19.3	26.7	UFS with LR	Drop

OK401	TDW	14.2	34.9	UFS with LR	Drop
OK402	TDW	14.2	34.9	UFS with LR	Drop
OK403	TDW	10.8	15.8	UFS with LR	Drop
DDG1	TDW	19.7	11.7	FC	(2)RC
DDG2	TDW	19.7	11.7	FC with LR	(2)RC
DDG3	TDW	19.7	11.7	UFS with LR and SRB	(2)RC
DDG4	TDW	19.7	11.7	UFS with LR	(2)RC

1 ft = 0.3048 mm
1 in = 25.4 mm

3.3 Testing Methods

This section will discuss key details of the test procedures used in both the laboratory and field investigation. The procedures have been separated into two categories: dynamic testing, and static testing. Dynamic testing includes all procedures which were intended to produce acceleration as the measured response. Static testing includes all procedures which were intended to produce deflection or force as the measured response. Further details can be found in Davis (2008).

3.3.1 Dynamic Testing

Floors were excited with impulse and harmonic forcing functions. Three accelerometers were used to measure the dynamic response of the floor, placed at the mid- and quarter-span points of the center joist, and at the mid-span point of the third joist from the edge. By instrumenting one quarter of the entire floor panel, damping and the first two natural frequencies were effectively captured. More data points would be needed to measure higher order natural frequencies and discern the mode shapes (Hanagan, Raebel, & Trethewey, 2003). The data acquisition hardware system is listed and discussed in Davis (2008).

The following two methods were used to provide the impulse excitation:

1. Sandbag (SB) drop: a 10 kg (22 lb) sandbag was dropped from 305 mm (12 in) above the floor at the geometric center. In the laboratory, the sandbag was suspended from an overhead crane so that the support and release mechanism did not interact with the floor system, as shown in Figure 3-4. In the field, a tripod was required, as shown in Figure 3-5. It was found that the natural frequencies from both the crane and tripod release mechanism produced very similar results (Davis, 2008). The sampling rate used was 500 Hz and the sample time was 10 s.
2. Heel drop (HD): a 82 kg (180 lb) individual stood at the center of the floor, rocked onto their toes, then dropped their heels to impact the floor with the force of their weight. This procedure, shown in Figure 3-6, was modeled on previous testing (Lenzen, 1966). This test was performed

identically in the laboratory and field. The sampling rate used was 500 Hz and the sample time was 10 s.

The sandbag drop was employed so that the measured frequency of the floor system was based on the mass of the structure only; the weight of an occupant is included when the heel drop is employed. The heel drop was employed so that the damping ratio could be determined with at least one occupant present on the floor. Occupant sensitivity may be affected by small changes in damping, and it has been shown that the inclusion of one or more occupants has a significant influence on damping ratio (Lenzen, 1966). At least one occupant must be on the floor to perceive any annoying vibrations, so one occupant was used to obtain the damping ratio. Also, the heel drop was employed so that the response of the floor could be compared to vibration acceptability criteria, or for a relative comparison between floor systems.

The heel drop is commonly used as the means of excitation for experimental floor vibration studies. Studies have shown that it does not replicate walking excitation, or isolate the modal properties of an unoccupied floor (Davis, 2008; Earnest, Fridley, & Cofer, 1998). For these reasons, it is suggested that the heel drop be used as a means of comparative evaluation only.

Figure 3-4: Sandbag Drop Test (Laboratory)

Figure 3-5: Sandbag Drop Test (Field)

Figure 3-6: Heel Drop Test

The harmonic excitation was provided by an 82 kg (180 lb) individual walking repeatedly across the floor. Two directions were measured individually: along mid-span, and along the center joist or center of the floor width. The sampling rate used was 100 Hz and the sample time was 50 s for each direction. The walking excitation was employed so that the response of the floor could be compared to vibration acceptability criteria.

3.3.2 Static Testing

A 1 kN (225 lb) load was placed at the geometric center of the floor, and the vertical joist deflection at mid-span, as well as the joist-end reactions were measured. In the laboratory, joist deflection was measured using analog dial gauges. The gauge probe contacted the bottom flange of the joist on the web side, as seen in Figure 3-7, so that flange curling was not measured. It was possible to measure the deflection for all floor systems and bearing conditions. Deflection was measured for each joist at mid-span. When the floor configuration included a ceiling, a small opening was cut to expose the bottom flange of the joist at mid-span. This procedure was also used for the DDG field floors. Figure 3-8 shows typical drywall openings, from a floor at the DDG location.

For field tests at the City Green location, deflection of the gypsum board ceiling was measured. Drywall openings were not permitted. The joist deflection was assumed to match the ceiling deflection within a reasonable amount error because the ceiling was attached directly to the joists. The 1 kN load was placed at the geometric center of the floor, as seen in Figure 3-9. Deflection was measured for the center three joists at mid-span, as seen in Figure 3-10. Joist deflection was not measured in field tests where the ceiling was suspended from the joists.

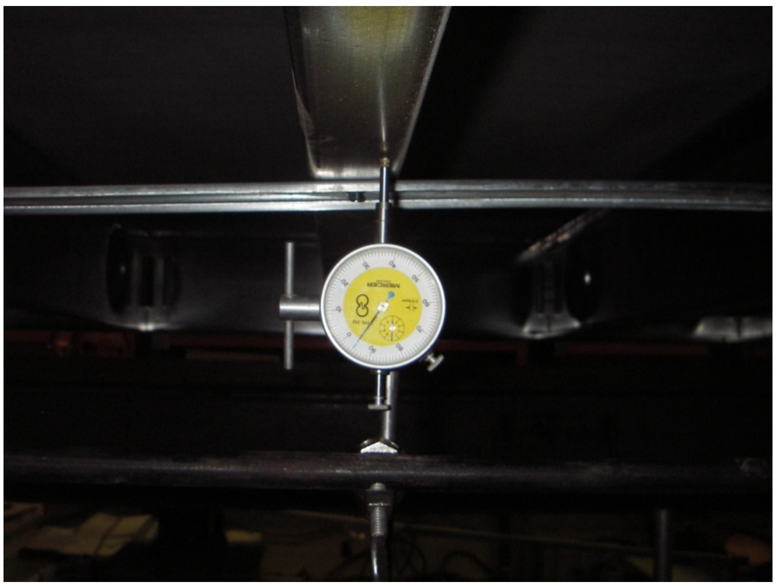

Figure 3-7: Joist Deflection Measurement

Figure 3-8: Drywall Openings for Joist Deflection Measurement

Figure 3-9: 1 kN Load from Static Test (Field)

Figure 3-10: Ceiling Deflection Measurement

It was possible to measure the joist-end reaction in the platform framing and simple support conditions in the laboratory, and not possible in the field whatsoever. All discussions related to the procedure and findings of the joist-end reaction tests are provided by Davis (2008), and not discussed in this thesis.

3.4 Data Analysis

The data analysis procedure used was based on previous studies conducted at the University of Waterloo, (Liu, 2001; Tangorra, 2005). The dynamic testing provided a raw time history of acceleration, which was processed to obtain natural frequencies, damping ratio, and root mean square (RMS) acceleration. Peak acceleration was read directly from the time history. Joist deflection was the average of three repeated tests.

3.4.1 Natural Frequencies

The natural frequencies were obtained by finding the frequency values which correspond to the dominant peaks in the power spectrum of the acceleration response from either impulse excitation. The power spectrum is a frequency domain plot which displays the frequency content of the time history (Hanagan, Raebel, & Trethewey, 2003). The power spectrum plot was obtained using the Fast Fourier Transform function in Matlab. Three accelerometers, located to measure multiple vibration modes, were used during the testing of the floor systems. The mean value of from all three accelerometers was reported for each type of excitation.

The first peak in the power spectrum corresponds to the fundamental frequency, which is generally associated with the first flexural mode. Work done by Johnson shows that higher-order bending and torsional modes contribute very little to floor response from impulse excitation (Johnson, 1994). It is common for all natural frequencies up to 40 Hz to be reported for floor vibration studies. However, discussion in this thesis is limited to the first two natural frequencies because they will have the greatest influence on the response of the floor system. Also, more accelerometer locations were required to successfully obtain higher order frequencies (Hanagan, Raebel, & Trethewey, 2003). The power spectrum shown Figure 3-11 is an example of an ideal plot; with the first two natural frequencies of approximately 10.5 Hz and 16.5 Hz.

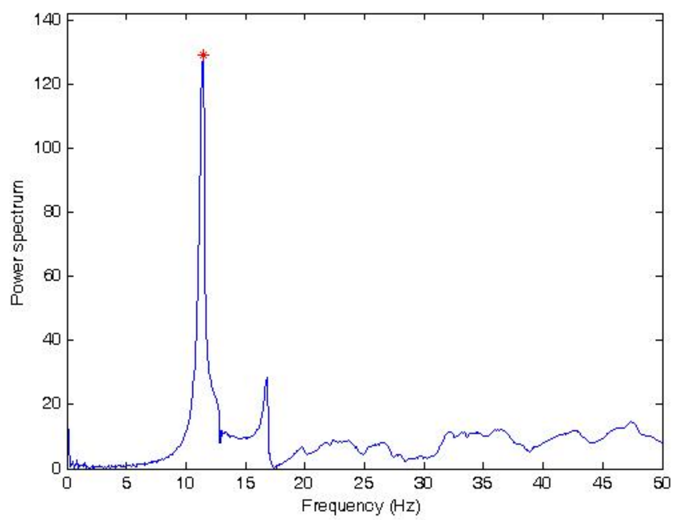

Figure 3-11: Example of Ideal Power Spectrum

This investigation focused on the fundamental and second natural frequencies only. Several test floors exhibited a large, flattened peak in the power spectrum, which made it difficult to obtain accurate values for the second and higher natural frequencies. For these cases (an example is provided in Figure 3-12), and where the second natural frequency was greater than 40 Hz, no value is reported. This procedure assumes that the excitation is an ideal impulse, which contains forcing contributions at all frequencies. It is the generally accepted method of determining natural frequencies of vibrating structures (Hanagan, Raebel, & Trethewey, 2003).

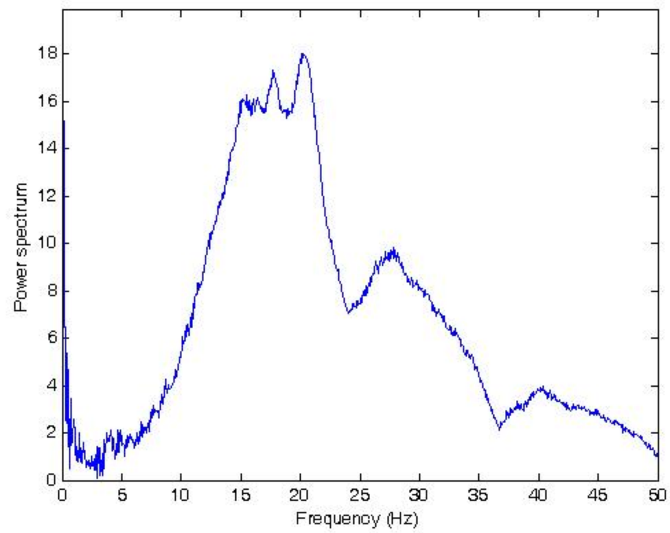

Figure 3-12: Example of Non-ideal Power Spectrum

3.4.2 Damping Ratio

Two different methods were used to determine the first modal damping ratio (ζ_1) for each floor system. The half-power bandwidth method was used to compute the damping ratio in the frequency domain, and the logarithmic decrement was used to compute the damping ratio in the time domain. Descriptions of these methods can be found in structural dynamics texts and other references (Davis, 2008; Liu, 2001).

The half-power bandwidth method can be used to find the modal damping ratio for each natural frequency of a system when that system is excited by an impulse load. The sandbag drop and heel drop were assumed to be impulse excitations, so the bandwidth method was applicable to the data from both of these tests. It has been shown that under an impulse excitation, the ith modal damping ratio (ζ_i) can be expressed as:

$$\zeta_i = \frac{\Delta f_i}{2 f_i} \qquad 3\text{-}1$$

where:

f_i is the ith natural frequency; and,

Δf_i is the bandwidth corresponding to the ith natural frequency (Liu, 2001).

This method is graphically illustrated in Figure 3-13. Bandwidth is defined as the line segment bounded by the rise and fall of the power spectrum at $\frac{1}{\sqrt{2}}$ times the maximum value.

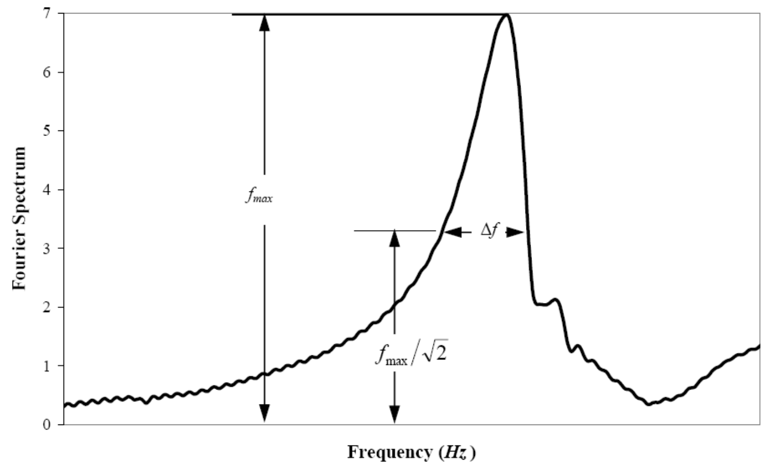

Figure 3-13: Half-Power Bandwidth Method (Tangorra, 2005)

The logarithmic decrement can be used to find the viscous damping ratio of a single-degree-of-freedom system under free vibration. Floors are continuous systems, but by applying a bandpass filter about the fundamental frequency, an equivalent single-degree-of-freedom system can be obtained. The filter used was a Butterworth bandpass filter, a built in function in Matlab, and a band of 1.5 Hz about the fundamental frequency was applied. The program used was based on a verified technique taken from previous research (Liu, 2001). The filtered result was treated as the response of a single-degree-of-freedom system in free vibration. The viscous damping ratio for the fundamental mode (ζ_o) can be obtained using the logarithmic decrement, which is expressed as the following:

$$\zeta_o = \frac{1}{2n\pi} \ln\left(\frac{A_i}{A_{i+n}}\right) \qquad \text{3-2}$$

where:

A_i is the amplitude at an arbitrary point; and,

A_{i+n} is the amplitude after n cycles.

It is recommended that A_i be taken five peaks away from the largest amplitude, which was not possible in several cases. The logarithmic decrement is reported for field floors where there were sufficient peaks in the filtered time series.

The half-power bandwidth method cannot obtain individual modal damping ratios for floor systems with closely spaced frequencies. Figure 3-11 shows a power spectrum plot where there is good modal separation, and the half-power bandwidth method applies. Figure 3-12 shows a power spectrum plot where there is poor modal separation, and the half-power bandwidth method does not apply. This phenomenon presented in several field test floors, and no value is reported for the damping ratio using this method.

3.4.3 Acceleration

Acceleration values reported from the heel drop test are the average of the peak values measured from each test. The reported RMS acceleration value from the walking test is greater acceleration from the two test directions. RMS acceleration was calculated using an adaptation of the procedure given in ISO 2631 that excluded the frequency weighting steps (ISO, 1997). The following expression is used to determine RMS acceleration (a_{RMS}):

$$a_{RMS} = \sqrt{\frac{1}{T}\int_0^T a(t)^2 dt} \tag{3-3}$$

where:

$a(t)$ is the acceleration time history; and,

T is the duration of the entire sample period.

Chapter 4. Experimental Results and Discussion

This Chapter contains experimental results from the laboratory and field investigations, and provides a discussion based on the observed results, expected behavior and the influence of specific details. An extensive discussion on the influence of construction details based on these experimental results can be found in Davis (2008). That discussion primarily focuses on how variations in detail can affect the fundamental frequencies and static deflection of floor systems. This thesis will expand on that discussion, and present a slightly modified data set, as is discussed subsequently.

4.1 Experimental Results

This section contains the experimental data, which is presented in Table 4-1 to Table 4-4. The tables contain the first and second natural frequencies, damping ratio(s), and center deflection for each floor system; as well as acceleration responses from heel drop, and walking excitation. Table 4-1 contains laboratory data in the balloon framing condition, Table 4-2 contains laboratory data in the platform framing condition, Table 4-3 contains laboratory data in the simple support condition, and Table 4-4 contains field testing data of the in situ floors, which are in the balloon framing condition.

Fundamental frequency is the average value obtained from the three accelerometers over all three iterations of the test, provided they are in general agreement. Results varying by 2.0 Hz or greater were excluded from the average. For this study, the fundamental frequency obtained using the sandbag drop procedure is presented in order to exclude the influence of the occupant mass required for the heel drop. This was done to facilitate comparison between measured and estimated fundamental frequency in Chapter 5. Using the natural frequencies obtained from the heel drop procedure, as provided in Davis (2008), will incorporate the mass of the occupant. Both fundamental frequency (f_1) and second natural frequency (f_2) are presented.

Two damping ratios are presented, if applicable: damping ratio calculated with the half-power bandwidth method (β_1), and damping ratio calculated with the logarithmic decrement (β_2). The methods were applicable provided that the data was suitable, based on the discussion in Section 3.4.2. If neither method was usable, the damping ratio is not provided. Damping ratio is based on the results of the heel drop test with the following exception: for laboratory floor systems without LEVELROCK topping or gypsum board ceiling (LF14.5A, LF14.5B, LF14.5C, and LF15.5D$_i$), the value obtained from the sandbag test was used. For these floors, the mass of the occupant for the heel drop was very high in comparison to the mass of the floor system, and the damping rations computed from these tests were unreasonably high, and excluded for this reason.

Center deflection (Δ_{center}) is the mean value obtained from the three measurements taken. Peak acceleration response (a_{peak}) is the average of the maximum acceleration values obtained for the three accelerometers, over all three iterations of the heel drop test. RMS acceleration response (a_{RMS}) is the maximum RMS acceleration computed for the three accelerometers, considering both directions tested.

4.1.1 Laboratory Investigation

Table 4-1: Experimental Data (Laboratory, Balloon Framing)

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	β_2 (%)	Δ_{center} (mm)	a_{peak} (g)	a_{RMS} (g)
LF14.5A	25.3	32.7	7.3%*	2.3%*	0.52	1.44	0.015
LF14.5B	22.5	25.1	2.3%*	3.0%*	0.44	1.19	0.016
LF14.5C	26.3	33.2	7.5%*	2.0%*	0.59	0.69	n/a
LF14.5D	19.7	24.2	4.7%	2.2%	0.34	1.34	0.014
LF14.5Di	24.1	28.8	3.8%*	2.1%*	0.38	2.05	0.022
LF14.5E	17.7	22.5	3.1%	2.3%	0.22	0.70	0.029
LF14.5F	16.1	22.5	3.8%	3.0%	0.18	0.72	0.037
LF17.0A	14.9	19.1	4.4%	4.0%	0.30	0.89	0.011
LF17.0B	14.9	19.7	3.9%	3.6%	0.27	0.78	0.012
LF17.0C	14.3	18.3	3.6%	3.0%	0.25	0.60	0.014
LF17.0D	14.3	19.9	3.4%	3.4%	0.22	0.68	0.013
LF19.5A	12.0	16.9	3.1%	2.9%	0.33	0.76	0.014
LF19.5A_{i}	12.7	17.4	4.4%	2.8%	0.37	0.84	n/a
LF19.5A_{ii}	13.2	24.0	4.5%	4.1%	0.35	1.01	n/a
LF19.5A_{iii}	13.0	23.0	4.5%	3.4%	0.30	0.86	n/a
LF19.5A_{iv}	10.4	14.8	2.6%	3.1%	0.33	0.56	n/a
LF19.5B	11.4	16.8	3.9%	3.4%	0.28	0.76	0.011
LF19.5B_{i}	12.0	17.5	3.0%	2.9%	n/a	0.92	n/a
LF19.5B_{ii}	12.7	25.0	4.3%	3.6%	0.29	0.93	n/a
LF19.5B_{iii}	12.5	23.4	4.0%	3.7%	0.26	0.75	n/a
LF19.5B_{iv}	10.0	14.9	2.4%	3.2%	n/a	0.62	n/a
LF21.8A	10.1	15.5	3.6%	3.3%	0.28	0.61	0.011

* indicates damping ratio from sandbag test

Table 4-2: Experimental Data (Laboratory, Platform Framing)

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	β_2 (%)	Δ_{center} (mm)	a_{peak} (g)	a_{RMS} (g)
LF14.5A	17.9	29.8	3.4%*	3.7%	0.67	n/a	0.013
LF14.5B	17.2	18.8	4.6%*	1.9%	0.48	1.26	0.014
LF14.5C	16.4	27.8	7.6%*	2.2%*	0.62	0.97	n/a
LF14.5D	16.9	22.0	7.0%	2.6%	0.37	1.52	0.012
LF14.5E	16.2	22.2	5.3%	3.9%	0.24	0.65	0.031
LF14.5F	14.8	22.0	3.4%	2.9%	0.17	0.60	0.039
LF17.0A	13.6	19.4	4.0%	3.4%	0.32	0.62	0.010
LF17.0B	13.3	19.3	5.7%	4.7%	0.29	0.69	0.011
LF17.0C	13.4	18.8	4.0%	3.5%	0.25	0.58	0.010
LF17.0D	13.4	20.2	4.1%	3.2%	0.23	0.58	0.010
LF19.5A	11.8	17.3	3.8%	3.2%	0.34	0.58	0.013
LF19.5A$_{iv}$	10.6	15.3	2.5%	2.2%	0.34	0.47	n/a
LF19.5B	11.1	17.0	3.5%	3.4%	0.28	0.74	0.014
LF21.8A	10.1	15.5	3.6%	3.5%	0.31	0.57	0.008

* indicates damping ratio from sandbag test

Table 4-3: Experimental Data (Laboratory, Simple Support)

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	β_2 (%)	Δ_{center} (mm)	a_{peak} (g)	a_{RMS} (g)
LF14.5A	19.1	27.4	6.7%*	3.0%	0.55	1.48	0.015
LF14.5B	17.2	21.4	3.7%*	2.8%	0.54	1.08	0.018
LF14.5B$_i$	16.4	24.8	2.4%	-	n/a	0.96	n/a
LF14.5C	17.7	26.0	7.8%*	1.7%*	0.71	0.93	n/a
LF14.5D	16.2	22.4	7.7%	2.4%	0.4	1.20	0.014
LF14.5E	15.7	21.1	5.7%	4.0%	0.25	0.56	0.029
LF14.5F	14.6	21.2	3.2%	3.1%	0.20	0.55	0.047
LF17.0A	13.5	17.9	4.8%	4.1%	0.34	0.72	0.011
LF17.0B	13.3	18.1	4.4%	3.4%	0.32	0.83	0.010
LF17.0C	12.8	18.4	3.2%	3.6%	0.26	0.63	0.014
LF17.0D	13.2	18.6	4.5%	3.3%	0.24	0.68	0.009
LF19.5A	11.4	16.4	4.9%	3.7%	0.35	0.64	0.010
LF19.5A$_{iv}$	10.1	14.7	3.5%	2.8%	0.35	0.52	n/a
LF19.5B	10.6	16.3	3.7%	3.8%	0.30	0.76	0.012
LF21.8A	9.9	14.5	3.6%	3.6%	0.34	0.53	0.009

* indicates damping ratio from sandbag test

Previously, values presented were a weighted average of both the logarithmic decrement and half-power bandwidth results (Davis, 2008). For this thesis, the values have been presented separately so that an expanded discussion can be made. The value presented is the first modal damping ratio, but it is treated as the overall viscous damping ratio of the structure for design purposes by some models. Most of the available models only have one degree of freedom, and assume only viscous damping is present. This does not produce significant errors for small damping values. Similar to the natural frequencies, the values from all accelerometers and test iterations are included in the average, if applicable.

4.1.2 Field Investigation

Table 4-4: Experimental Data (In Situ, Balloon Framing)

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	β_2 (%)	Δ_{center} (mm)	a_{peak} (g)	a_{RMS} (g)
CG601	14.4	23.1	-	-	0.46	0.74	0.007
CG604	16.3	23.1	15.2%	-	0.28	0.70	0.006
CG805	15.2	22.8	8.0%	8.2%	n/a	0.57	n/a
CGMH6	15.7	23.8	11.3%	8.8%	0.36	0.71	n/a
CGMH7	16.6	25.6	12.3%	7.1%	n/a	1.15	n/a
CW707	16.1	20.2	7.3%	9.8%	n/a	0.45	0.003
CW708	18.7	22.9	7.3%	7.7%	n/a	0.37	0.005
CW709	9.9	12.9	5.8%	-	n/a	0.35	0.005
CW805	11.9	23.0	12.6%	-	n/a	0.37	n/a
DDG1	12.0	21.6	11.8%	8.9%	0.39	1.10	0.014
DDG2	14.0	-	10.8%	-	0.37	0.54	0.006
DDG3	13.2	27.1	8.4%	-	0.2	0.66	0.008
DDG4	16.1	-	6.1%	-	0.08	0.45	0.005
OK401	22.3	28.6	4.5%	-	n/a	0.67	0.004
OK402	23.7	27.9	6.0%	-	n/a	0.43	0.003
OK403	32.8	45.7	-	-	n/a	0.72	0.002

4.2 Discussion of Results

The major focus of the research by Davis was the investigation of the influence of several variations of construction details on the modal properties of the laboratory and floor systems. The following conclusions were made (Davis, 2008):

- Fundamental frequency and center deflection tends to increase if span length is increased. This is because beam bending stiffness reduces with increasing span, and the floor weight increases.

- Use of joists with large, lip-reinforced openings (based specifically on the opening design and spacing found on the TradeReady joists) does not appear to influence the dynamic performance of a floor system. This is because the bending properties of the cross-section are similar at service loads. These results cannot be extended to joists with closely-spaced openings.
- Bare FORTACRETE subfloors will have lower fundamental frequencies and less center deflection than equivalent bare OSB floors. This is because FORTACRETE is stiffer and significantly heavier than OSB.
- UFS subfloors with LevelRock topping have a slightly lower fundamental frequency than FORTACRETE subfloors with LevelRock topping. The UFS assembly also has less center deflection and better load sharing capabilities. The UFS assembly is more massive and has greater transverse stiffness, which accounts for this behavior. These observations may not be applicable to floor systems with topping thicknesses that vary from the test details.
- The addition of a gypsum board ceiling supported by resilient channel reduces the fundamental frequency and center deflection of a floor system. The ceiling adds mass to the assembly, and the resilient channel and gypsum board appear to increase continuity between the joists. These tests were based on the balloon framing condition only. It is cautioned that similar stiffening effects will not be observed in floors with drop ceilings. The influence of adding a ceiling on the damping ratio was not observed based on the data presented in Davis (2008).
- The addition of a strongback with unrestrained ends reduces the center deflection of a floor system. The unrestrained strongback adds transverse stiffness to the floor system without affecting primary bending stiffness.
- The addition of a strongback with restrained ends increases the fundamental frequency and damping ratio, and reduces the center deflection of a floor system. The restrained strongback resists local movement of the floor at is support points. This constraint increases fundamental frequency and accounts for the reduction in center deflection. It is not understood why this condition would provide an observable change in damping while the free end condition did not. A potential explanation is that the strongback dissipated energy through bending and transmission to the supports. These observations cannot be specifically applied to constructed floors because testing was not done to determine the criteria required to provide effective end restraint for a strongback.
- The addition of a superimposed live load reduces the fundamental frequency of a floor system without increasing the damping ratio. Additional mass may not increase damping in a floor system, unless it is due to the addition of materials with inherent damping properties.

- The balloon end-framing condition results in the highest fundamental frequency and least center deflection for a floor system. This indicates that balloon framing provides the greatest end restraint.
- The simple-support end-framing condition generally results in the highest damping ratio for a floor system, while providing the least end restraint. Energy is dissipated by the hysteretic behavior of the floor vibrating on the supports if no rotational restraint, from balloon framing or superimposed load, is present.

4.2.1 Transverse Stiffness

It is generally assumed that the fundamental mode of vibration for a floor system is one-way primary bending, which takes on the mode shape of a half-sine wave (Allen & Murray, 1993). A more accurate depiction would be an orthotropic place which is supported on all four sides (Smith & Chui, 1988). The orthotropic plate model is significantly more complicated to compute and does not necessarily constitute an improved model for lightweight floor systems, as is shown in Chapter 5. A floor must have a significant amount transverse stiffness for the edge conditions to affect the flexibility at the geometric center.

The fundamental frequency of a floor system is influenced by the transverse stiffness because it controls the effective floor mass. The second natural frequency is also controlled by the transverse stiffness of the floor system; higher transverse stiffness increases the separation between the first and second modes (Allen, Onysko, & Murray, 1999). The second mode of vibration is typically a torsional mode, involving a full-sine wave in the transverse direction at mid-span, and opposite half-sine waves in the joist direction. These are general assumptions, and are dependent on several factors including aspect ratio and relative stiffness.

To this effect, construction details which increase transverse stiffness are expected to increase the separation between the f_1 and f_2. Details that were found to influence transverse stiffness include subfloor assembly, ceilings, strongbacks and end-restraint conditions. The following tables contain a list of data which illustrate the transverse stiffness behavior of the laboratory floor systems:

Table 4-5 illustrates the variation of subfloor material, Table 4-6 illustrates the addition of a strongback with free and fixed ends, and Table 4-7 illustrates the addition of a gypsum board ceiling. The aspect ratio of the floor systems (length divided by width) is included so that the floors can be classified as narrow, square, and long. The relative frequency separation (the difference between first and second frequencies, divided by the first) is a normalized expression to indicate the separation between the fundamental frequency and second natural frequency. A greater relative or absolute separation can be attributed to increased transverse stiffness. The influence of subfloor, strongback and ceiling are discussed separately. The following discussion will outline qualitative observations because the limited data set does not lend itself to numerical assessment.

Table 4-5: Transverse Stiffness Comparison (Subfloor)

Framing Condition	Floor Name	Aspect Ratio	Varied Detail	f_1 (Hz)	f_2 (Hz)	$\Delta f/f_1$	Δ_{center} (mm)
Balloon	LF14.5A	0.91	OSB	25.3	32.7	0.29	0.52
Balloon	LF14.5C	0.91	OSB	26.3	33.2	0.26	0.59
Balloon	LF14.5B	0.91	FC	22.5	25.1	0.12	0.44
Balloon	LF14.5D_i	0.91	FC	24.1	28.8	0.20	0.38
Simple	LF14.5A	0.91	OSB	19.1	27.4	0.43	0.55
Simple	LF14.5C	0.91	OSB	17.7	26.0	0.47	0.71
Simple	LF14.5B	0.91	FC	17.2	21.4	0.24	0.54
Platform	LF14.5A	0.91	OSB	17.9	29.8	0.66	0.67
Platform	LF14.5C	0.91	OSB	16.4	27.8	0.70	0.62
Platform	LF14.5B	0.91	FC	17.2	18.8	0.09	0.48
Balloon	LF14.5E	0.91	FC with LR	17.7	22.5	0.27	0.22
Balloon	LF14.5F	0.91	UFS with LR	16.1	22.5	0.40	0.18
Simple	LF14.5E	0.91	FC with LR	15.7	21.1	0.34	0.25
Simple	LF14.5F	0.91	UFS with LR	14.6	21.2	0.45	0.20
Platform	LF14.5E	0.91	FC with LR	16.2	22.2	0.37	0.24
Platform	LF14.5F	0.91	UFS with LR	14.8	22.0	0.49	0.17
Balloon	LF17.0A	1.06	FC with LR	14.9	19.1	0.28	0.3
Balloon	LF17.0C	1.06	UFS with LR	14.3	18.3	0.28	0.25
Simple	LF17.0A	1.06	FC with LR	13.5	17.9	0.33	0.34
Simple	LF17.0C	1.06	UFS with LR	12.8	18.4	0.44	0.26
Platform	LF17.0A	1.06	FC with LR	13.6	19.4	0.43	0.32
Platform	LF17.0C	1.06	UFS with LR	13.4	18.8	0.40	0.25
Balloon	LF19.5A	1.22	FC with LR	12.0	16.9	0.41	0.33
Balloon	LF19.5B	1.22	UFS with LR	11.4	16.8	0.47	0.28
Simple	LF19.5A	1.22	FC with LR	11.4	16.4	0.44	0.35
Simple	LF19.5B	1.22	UFS with LR	10.6	16.3	0.54	0.30
Platform	LF19.5A	1.22	FC with LR	11.8	17.3	0.47	0.34
Platform	LF19.5B	1.22	UFS with LR	11.1	17.0	0.53	0.28

Table 4-5 contains data from several changes in subfloor material; with identical joists, spans and other details. For this table, and all subsequent tables in this section, the thin lines separated floors which are directly compared, while the thick lines separate the floors by aspect ratio. The aspect ratio is of particular importance in this section, as the observations vary for short (aspect ratio of 0.91), nearly square (aspect ratio of 1.06), and long floors (aspect ratio of 1.22). The following observations were made on the variation of subfloor assemblies:

- For short floors, bare OSB exhibits twice the relative separation of bare FORTACRETE but significantly less center deflection. This is counterintuitive because the transverse stiffness of bare FORTACRETE is over two times that of OSB (see Appendix C).
- This observation may be explained by specifically evaluating the selection of natural frequencies of the OSB floor. Figure 4-1 shows the power spectrum of the response at one accelerometer from the sandbag drop test on LF14.5A, where the identified peaks are indicated at 27, 32, and 40 Hz. The peak at 27 Hz was initially selected because it is the most dominant peak, but there is a smaller peak at approximately 25 Hz that can be also identified. Given the expectation of the behavior of the OSB subfloor, it is possible that the first and second natural frequencies could be spaced this closely together, but from the data presented it appears that the peak at 27 Hz was not identified. The relative frequency separation values for bare OSB should, therefore, be excluded from further discussion.
- For both short and long floors, UFS with LEVELROCK exhibits greater relative separation than FORTACRETE with LEVELROCK and has less center deflection. This suggests that the UFS subfloor assembly provides increased transverse stiffness, which agrees with the fact that the UFS subfloor has over two times the bending stiffness in the transverse direction (see Appendix C). For the nearly square floor, no trend in relative separation was observed, indicating that aspect ratio may play a role in determining the mode shapes of a floor system.

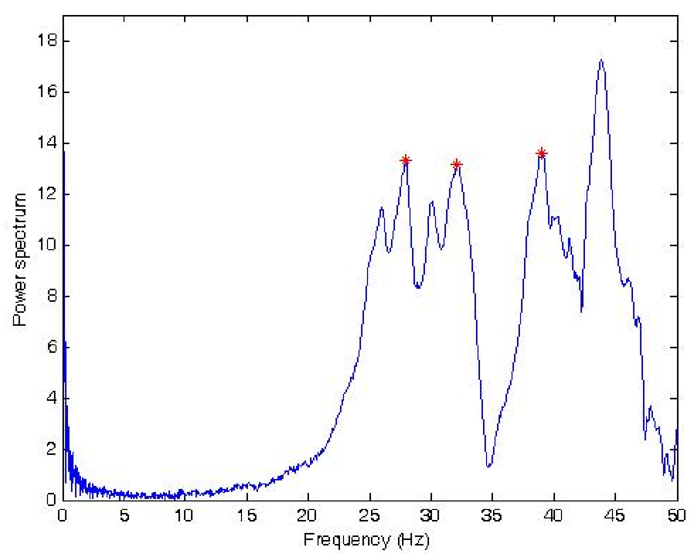

Figure 4-1: LF14.5A Frequency Selection

Presented in Table 4-6 is the influence on transverse stiffness from the addition of a strongback with either unrestrained or restrained ends; with identical joists, spans, subfloors, and other details. The following observations were made on the variation of strongback details:

- For nearly square floors, where the unrestrained strongback was added, relative separation was decreased in most cases. Fundamental frequency was not affects and center deflection was only reduced in some cases, despite the increased relative separation of the first two frequencies. This suggests that the strongback member alone has little impact on the overall transverse stiffness of the floor, but some improvement can be expected.
- For long floors, where the restrained strongback was added, relative separation experienced a considerable increase in all cases; along with a slight increase in fundamental frequency, and slight reduction in center deflection. This suggests that the strongback and associated edge restraint have a significant impact on both transverse bending stiffness of the floor, as well as imposing constraints in the primary bending direction.

Table 4-6: Transverse Stiffness Comparison (Strongback)

Framing Condition	Floor Name	Aspect Ratio	Varied Detail	f_1 (Hz)	f_2 (Hz)	$\Delta f / f_1$	Δ_{center} (mm)
Balloon	LF17.0A	1.06	-	14.9	19.1	0.28	0.30
Balloon	LF17.0B	1.06	SB (Free)	14.9	19.7	0.32	0.27
Balloon	LF17.0C	1.06	-	14.3	18.3	0.28	0.25
Balloon	LF17.0D	1.06	SB (Free)	14.3	19.9	0.39	0.22
Simple	LF17.0A	1.06	-	13.5	17.9	0.33	0.34
Simple	LF17.0B	1.06	SB (Free)	13.3	18.1	0.36	0.32
Simple	LF17.0C	1.06	-	12.8	18.4	0.44	0.26
Simple	LF17.0D	1.06	SB (Free)	13.2	18.6	0.41	0.24
Platform	LF17.0A	1.06	-	13.6	19.4	0.43	0.32
Platform	LF17.0B	1.06	SB (Free)	13.3	19.3	0.45	0.29
Platform	LF17.0C	1.06	-	13.4	18.8	0.40	0.25
Platform	LF17.0D	1.06	SB (Free)	13.4	20.2	0.51	0.23
Balloon	LF19.5A	1.22	-	12.0	16.9	0.41	0.33
Balloon	LF19.5A$_{iii}$	1.22	SB (fixed)	13.0	23.0	0.77	0.30
Balloon	LF19.5B	1.22	-	11.4	16.8	0.47	0.28
Balloon	LF19.5B$_{iii}$	1.22	SB (fixed)	12.5	23.4	0.87	0.26
Balloon	LF19.5A$_{i}$	1.22	-	12.7	17.4	0.37	0.37
Balloon	LF19.5A$_{ii}$	1.22	SB (fixed)	13.2	24.0	0.82	0.35
Balloon	LF19.5B$_{i}$	1.22	-	12.0	17.5	0.46	-
Balloon	LF19.5B$_{ii}$	1.22	SB (fixed)	12.7	25.0	0.97	0.29

Presented in Table 4-7 is the influence of the addition of a gypsum board ceiling attached through resilient channel; with identical joists, spans, subfloors, and other details. The following observations were made on the variation of ceiling details:

- For short floors, adding a ceiling increases the relative separation of frequencies. Fundamental frequency is reduced, while center deflection is less. This indicates that the ceiling has little impact on the primary bending stiffness, while acting to increase the transverse stiffness of the floor system.
- For long floors without a strongback detail, adding a ceiling provides only a marginal increase in relative separation, if any at all. A similar outcome was found for reduction in center deflection.
- For long floors with a strongback detail, adding a ceiling was found to reduce relative separation. This indicates that the transverse stiffening properties from the ceiling and restrained strongback are not additive. An effective strongback appears to reduce the contribution of a ceiling to mass only.

Table 4-7: Transverse Stiffness Comparison (Ceiling)

Framing Condition	Floor Name	Aspect Ratio	Varied Detail	f_1 (Hz)	f_2 (Hz)	$\Delta f/ f_1$	Δ_{center} (mm)
Balloon	LF14.5D	0.91	Ceiling	19.7	24.2	0.23	0.34
Balloon	LF14.5D$_i$	0.91	-	24.1	28.8	0.20	0.38
Balloon	LF19.5A	1.22	Ceiling	12.0	16.9	0.41	0.33
Balloon	LF19.5A$_i$	1.22	-	12.7	17.4	0.37	0.37
Balloon	LF19.5A$_{ii}$	1.22	-	13.2	24.0	0.82	0.35
Balloon	LF19.5A$_{iii}$	1.22	Ceiling	13.0	23.0	0.77	0.30
Balloon	LF19.5B	1.22	Ceiling	11.4	16.8	0.47	0.28
Balloon	LF19.5B$_i$	1.22	-	12.0	17.5	0.46	-
Balloon	LF19.5B$_{ii}$	1.22	-	12.7	25.0	0.97	0.29
Balloon	LF19.5B$_{iii}$	1.22	Ceiling	12.5	23.4	0.87	0.26
Simple	LF19.5A	1.22	Ceiling	11.4	16.4	0.44	0.35
Simple	LF19.5A$_{iv}$	1.22	-	10.1	14.7	0.46	0.35
Platform	LF19.5A	1.22	Ceiling	11.8	17.3	0.47	0.34
Platform	LF19.5A$_{iv}$	1.22	-	10.6	15.3	0.44	0.34

When a designer desires to increase the fundamental frequency of a floor system, it has been shown that increasing primary bending stiffness is the best approach. The most efficient way to increase fundamental frequency is to increase the moment of inertia of the joists, which adds bending stiffness and relatively little mass. If remediation is desired, an effectively restrained strongback will improve both fundamental frequency and transverse bending stiffness, and exceed the performance improvement of adding a gypsum board ceiling. When a strongback and ceiling are used together, it should not be assumed that the ceiling will contribute to additional transverse stiffness.

4.2.2 Damping Ratio

Damping ratio is the modal property which has the most influence on how occupants perceive floor vibrations, as well as affecting the magnitude of the dynamic response of a floor to any excitation. Sufficient damping will ensure that the period of free vibration is small enough that walking vibrations are perceived as transient excitation, which can reduce occupant sensitivity by a factor of 10 over harmonic excitation (Lenzen, 1966). A quicker rate of decay will also reduce the acceleration response. Damping is an inherent property of the materials, construction methods, and dimensions of a floor system, as well as the type of occupancy. It is very difficult to predict the damping ratio of a floor system in the design stage, even if only a few variables are considered (Allen, Onysko, & Murray, 1999). A suitable damping ratio must be assumed by designers, thus warranting an extended discussion on factors affecting damping ratio and recommended design values.

Two methods were used to estimate the first modal damping ratio of six field floor systems: the half-power bandwidth method, listed as β_1; and, the logarithmic decrement, listed as β_2. Table Table 4-8 contains the damping ratio from both methods for all applicable floors. Listed are the corresponding subfloor assemblies, aspect ratios and floor weight in order to observe any trends. Aspect ratio and weight do not appear to have any influence on the effectiveness of either estimation method. For the UFS subfloors, the half-power bandwidth method provides a lower damping ratio than the logarithmic decrement. This could be explained by the extra separation between modes provided by the greater transverse stiffness of the UFS assembly, which will provide a power spectrum similar to the one shown in Figure 3-11, resulting in a very accurate estimate.

The least of the two values should be taken as the modal damping ratio. The values over 10% were obtained when peak interference was observed in the frequency domain plot, and have been excluded. The standard deviation of the absolute difference between β_1 and β_2 is 1.9%, which is assumed to be the potential error by using only one method alone, and can be applied to the laboratory and field testing results where β_1 is used exclusively.

Table 4-8: Comparison of Damping Ratio Calculation Methods

Floor Name	β_1 (%)	β_2 (%)	Subfloor Assembly	Aspect Ratio	Weight (kN/m)
CG805	8.0	8.2	FC with LR and SRB	0.76	0.46992
CGMH6	11.3	8.8	FC with LR and SRB	0.71	0.41301
CGMH7	12.3	7.1	FC with LR and SRB	0.71	0.41301
CW707	7.3	9.8	UFS with LR	0.65	0.4546
CW708	7.3	7.7	UFS with LR	0.51	0.4546
DDG1	11.8	8.9	FC	1.68	0.18272

A recommended design damping ratio of 6.0% for the design of residential floor systems supported by cold-formed steel joists is recommended by Davis (Davis, 2008). This value was taken from a combined data set including field and laboratory testing, disregarding the construction detail differences between floor systems. This approach was taken because very few details were found to impact damping.

After re-examining the methods used to calculate damping ratio, a detailed investigation was conducted in order to determine if appropriate design damping ratios could be recommended for the floor systems with the following subfloor assemblies: bare FORTACRETE, FORTACRETE with 0.75" (19.5 mm) LevelRock topping, and UFS with 38.1 mm (1.5 in) LevelRock topping. The data set used for the recommended values is a combination of the laboratory floor systems in the balloon framing condition and all field floor systems. Only floors with ceilings and without strongback details were included in order to provide the most general recommendations. Most of the field floor systems with the FORTACRETE and LevelRock assemblies also had SRB in the assembly, so a recommended increase in damping from SRB is discussed.

Table 4-9 contains recommended design damping ratio values which are based on the examination of various subfloor assemblies. Because the balloon framing values were used from the laboratory testing, the values will be lower than if the design uses platform framing, but this was not confirmed with in situ testing. The first row is a recommended β for design, based on averages taken over the data set. Damping ratios of over 10% were excluded, and the minimum was taken where both damping ratio calculations were used. The spread in values represents approximately one standard deviation on either side of the mean. The bare FORTACRETE subfloor assembly was tested in two floor systems only, so the recommended value is the minimum of the two tests and should not be treated as conclusive. It is shown that both subfloor assemblies with the LevelRock topping have similar damping values. The lower values in the range should be selected when the floor system supports a relatively empty room (such as a paperless office) without fixed partitions. The higher values should be selected when there are many partitions on the floor system, which can be found in some residential designs.

The second row in Table 4-9 lists suggested modifications based on additional details in the floor system. The inclusion of SRB in the subfloor assembly significantly increased the damping ratio of the floor systems, and an increase of 2.5% over the subfloor assembly value is recommended. An effectively restrained strongback increased damping in some of the laboratory tests and an additional 1.0% of damping is suggested, but no increase was found for floors with the UFS and LevelRock subfloor assembly. This suggestion is based on two data points only. Finally, the maximum damping difference between floor systems with and without a ceiling was found to be 1.5%, which is the suggested reduction for design if a ceiling is not included. In some cases, a ceiling was found to reduce damping ratio, but since the recommended values are based on in situ testing results with a ceiling already in place, any reduction is already accounted for in the measured damping. This reduction cannot be extended to a suspended ceiling detail.

Table 4-9: Recommended Design Damping Ratios by Subfloor Assembly

Subfloor Assembly	**FC**	**FC with LR**	**UFS with LR**
Design Damping Ratio	> 4.5%	3.5 to 6.0%	4.0 to 6.5%
Additional Detail	**Add SRB**	**Add Strongback**	**No Ceiling**
Additional Design Damping	+2.5%	+1.0% (+0.0% for UFS with LR)	-1.5%

In this study, no values are provided for the OSB subfloor for three reasons: no in situ tests were conducted, there may have been an error in the peak selection, and the OSB subfloor was glued. No testing was conducted to investigate the influence of a glued subfloor on the damping of a floor system, but previous testing has shown that it will decrease the damping ratio (Tangorra, 2005). Because FORTACRETE and OSB are of similar dimension and installation, it is expected that gluing the FORTACRETE panels to the floor joists, along with screwing them, will reduce the damping ratio to some degree. This is not a recommended construction practice for floor systems which are sensitive to the amount of damping present.

Observations from the Ocean Keys development provide an excellent example of the impact of damping ratio on the dynamic response of a floor system. The floors labeled OK401 and OK402 were constructed with identical floor plans, the difference being that OK402 was supported by an exterior wall on one side, while OK401 was supported by two interior walls. The two floor systems have similar fundamental frequencies, but OK402 was observed to have 1.5% more damping than OK401. Peak acceleration (a_{peak}) response from a heel drop and the RMS acceleration response from walking (a_{RMS}) are compared in Table 4-10. OK402, with greater damping, has a significantly lower peak response and marginally lower RMS response. This illustrates the importance of both selecting an appropriate damping ratio for design, and providing enough damping in the floor system to reduce annoying accelerations.

Table 4-10: Effect of Damping on Acceleration Response in Similar Floors

Floor Name	**f_1 (Hz)**	**β (%)**	**a_{peak} (g)**	**a_{RMS} (g)**
OK401	22.3	4.5%	0.67	0.0035
OK402	23.7	6.0%	0.43	0.0033

Chapter 5. Vibration Serviceability

This Chapter contains discussion of the vibration serviceability performance of in situ floor systems, and an assessment of several design guide methods. Modifications are proposed to the design guide methods in order to increase their accuracy and applicability for use with cold-formed steel floor systems. Recommendations for future testing in order to obtain material-specific coefficients are given, and suggestions for improvements in floor testing equipment and procedures are made.

5.1 Vibration Serviceability Performance of In Situ Floor Systems

The actual design and performance of the in situ floor systems is evaluated based on the following: relative comparison based on heel-drop excitation, the ISO Acceleration Limit for residential occupancy, and Onysko's static stiffness criterion defined by the ATC Design Guide (Allen, Onysko, & Murray, 1999).

5.1.1 Relative Comparison

In order to compare the relative performance of the in situ floor systems under heel drop excitation, the fundamental frequency must be considered. Because occupant sensitivity decreases with fundamental frequency, above 8 Hz, the following weighting was applied to the responses:

$$a_w = \left(\frac{8}{f_o}\right) a_{peak} \tag{5-1}$$

where:

a_{peak} is some maximum acceleration value; and,

a_w is the frequency-weighted acceleration response.

This expression is taken from Smith and Chui (1988), and along with relative comparisons the responses can be compared against their recommended limit of 0.45 m/s^2. The results are presented in Table 5-1. There is a large variation in a_w presented, the greatest value being approximately 4 times the least value. None of the floor systems come close to achieving the limit of 0.45 m/s^2 which suggests that the heel drop excitation used for the testing may have been of greater magnitude than the one studied by Smith and Chui. The limit in their method is based on an estimated response to an idealized design heel drop. In a response to this article, Allen and Rainer commented that the heel drop is not an appropriate measure of floor performance because it does not represent real service excitation (1989).

Table 5-1: Frequency Weighted Heel Drop Response

Floor Name	a_{peak} (m/s^2)	f_o (Hz)	a_w (m/s^2)
CG601	7.21	13.6	4.24
CG604	6.82	15.4	3.54
CG805	5.58	14.4	3.10
CGMH6	6.91	15.0	3.69
CGMH7	11.32	15.4	5.88
CW707	4.45	16.1	2.21
CW708	3.65	18.7	1.56
CW709	3.38	9.9	2.73
CW805	3.60	11.8	2.44
DDG1	10.80	11.9	7.26
DDG2	5.33	13.2	3.23
DDG3	6.42	13.1	3.92
DDG4	4.43	16.1	2.20
OK401	6.61	17.8	2.97
OK402	4.19	16.6	2.02

5.1.2 ISO Acceleration Response

The maximum RMS acceleration response measured for each floor system is plotted against the ISO Acceleration Limit for Residential Occupancy. The RMS acceleration is calculated using Equation 3-3. The plot in Figure 5-1 shows the measured in situ floor system response with the limiting residential criterion as a dashed line.

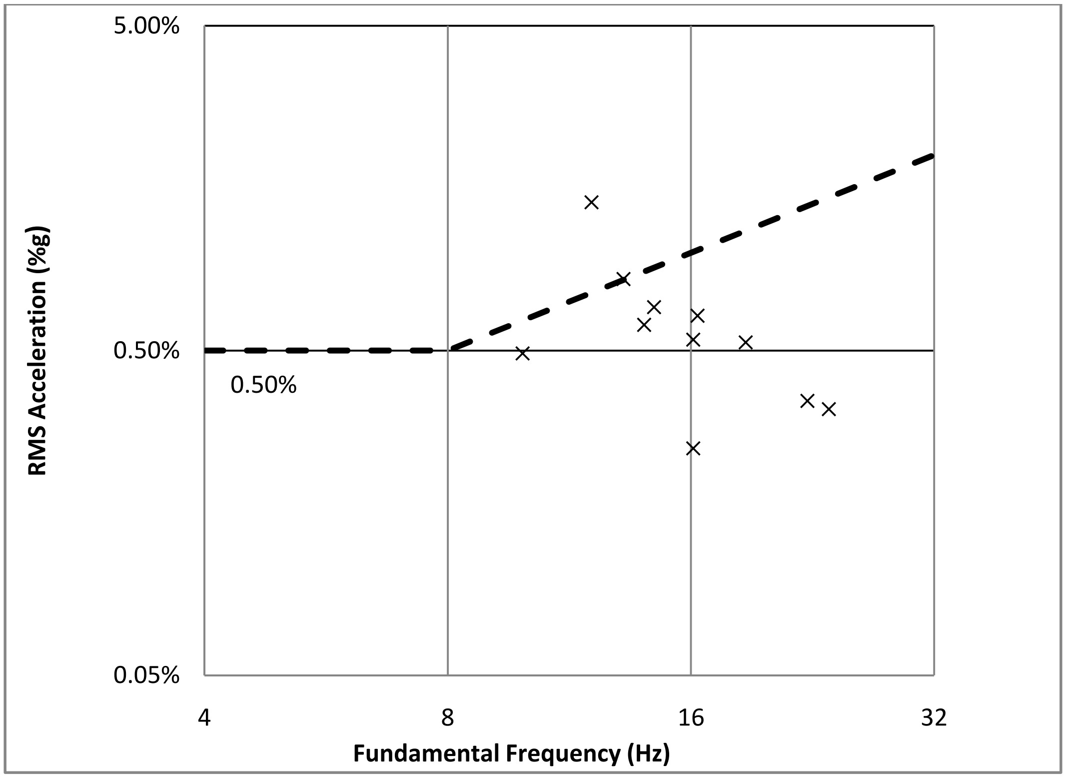

Figure 5-1: ISO Acceleration Criterion with In Situ Floors

All but two of the in situ floor systems tested were found to be below the maximum allowable acceleration specified by ISO. The measured values and limiting acceleration are provided in Table 5-2. It can be seen that DDG1 exceeded the allowable acceleration by almost two times, while DDG3 had exactly the maximum allowable acceleration. DDG1 had a bare FORTACRETE subfloor assembly, which was the only floor system in tested in the field without topping. This should not be interpreted as evidence that floor systems without topping will have unsatisfactory vibration acceptability performance. This particular case was a long, narrow floor without any partitions installed, and the resulting fundamental frequency was significantly lower than the floor systems without topping that were tested in the laboratory portion of this study. There was a double layer of gypsum board ceiling present, which lowered the fundamental frequency without significantly contributing to the stiffness of the system. DDG2 had a FORTACRTE and LEVELROCK subfloor assembly, and exhibited both a lower RMS acceleration response and higher fundamental frequency. Because both DDG1 and DDG3 were located in the sound test chambers, it can be concluded that all the in situ floor systems tested in residential buildings met the ISO maximum acceleration criterion for residential occupancy.

Table 5-2: ISO Acceleration Criterion with In Situ Floors

Floor Name	a_p/g measured	a_o/g ISO maximum
CG601	0.0068	0.0090
CG604	0.0064	0.0102
CG805	-	0.0095
CGMH6	-	0.0098
CGMH7	-	0.0104
CW707	0.0025	0.0101
CW708	0.0053	0.0117
CW709	0.0049	0.0062
CW805	-	0.0074
DDG1	0.0143	0.0075
DDG2	0.0060	0.0088
DDG3	0.0083	0.0083
DDG4	0.0054	0.0101
OK401	0.0035	0.0139
OK402	0.0033	0.0148
OK403	0.0022	0.0205

5.1.3 Onysko's Static Stiffness Criterion

The center deflection under a 1 kN point load was measured for the in situ floor systems at the DDG and CG locations. Measurements at the DDG location were possible because openings were cut into the ceilings in the sound test chambers. Measurements at the CG location were for deflection of the gypsum board ceiling, which was attached with resilient channel. At the CW and OK sites, the ceiling was suspended and deflection measurements were not possible.

The static deflection criterion applied is a modification on Onysko's original proposed limit, which is published in the ATC Design Guide 1 (Allen, Onysko, & Murray, 1999). The limit for deflection under a 1 kN point load (Δ_p), in inches, is given by the following expression:

$$\Delta_p = 0.024 + 0.1e^{-0.18(L-6.4)} \leq 0.08\ in \qquad \text{5-2}$$

where L is the span of the joists (in feet). This limit, converted to SI units, is plotted as a solid line against the measured deflections from the in situ floor systems in Figure 5-2.

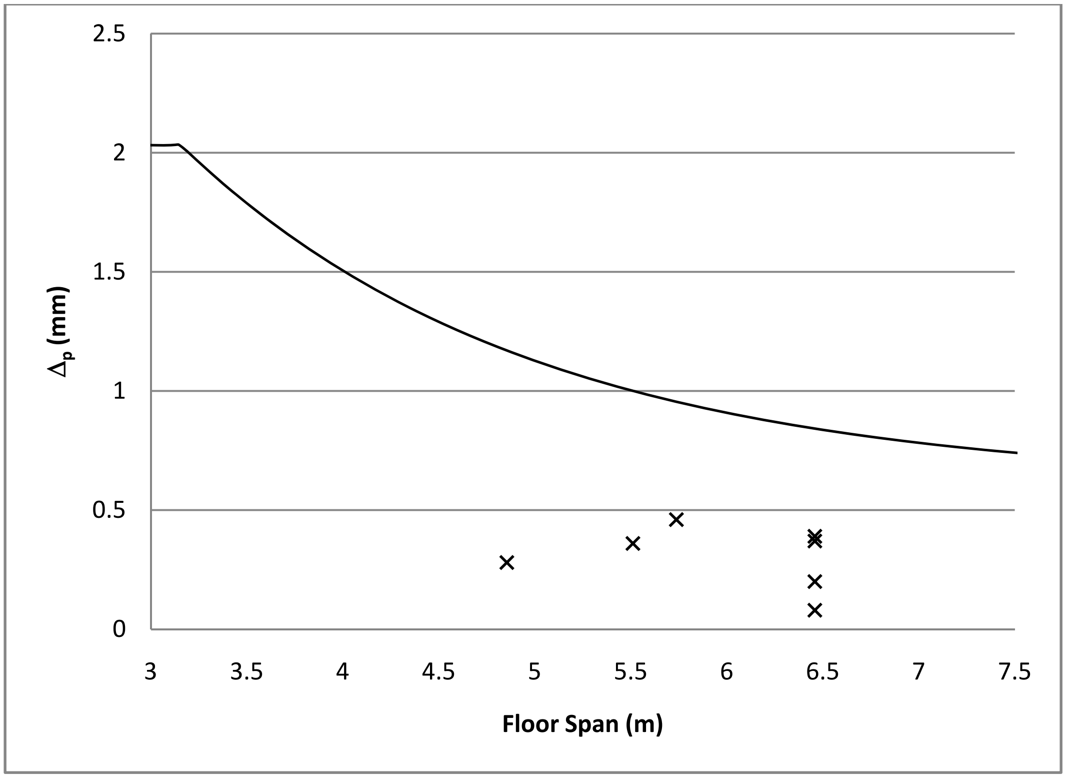

Figure 5-2: Onysko's Static Stiffness Criterion with In Situ Floors

All of the center deflections observed from the in situ floors were found to be well below the prescribed limit. Similarly, when this limit was applied to the laboratory test floors, all of the measured deflections were significantly below the limit (Davis, 2008). The measured deflections and associated limits for the span of each in situ floor tested are provided in Table 5-3.

Table 5-3: Onysko's Static Stiffness Criterion with In Situ Floors

Floor Name	Δ_p (mm) measured	Δ_p (mm) Onysko Maximum
CG601	0.46	0.95
CG604	0.28	1.17
CGMH6	0.36	1.00
DDG1	0.39	0.84
DDG2	0.37	0.84
DDG3	0.20	0.84
DDG4	0.08	0.84

5.2 Application of Vibration Design Methods to Floor Systems Supported by Cold-Formed Steel Joists

The following section contains an examination of the applicability of several design methods for assessing vibrations serviceability performance of floor systems supported by cold-formed steel joists by discussing the idealized models used, and evaluating the prediction of fundamental frequency, center deflection and RMS acceleration response. The following design methods are assessed: ATC Design Guide 1: Minimizing Floor Vibration (Allen, Onysko, & Murray, 1999); AISC/CISC Steel Design Guide Series #11: Floor Vibrations Due to Human Activity (Murray, Allen, & Ungar, 1997); and, Smith, Chui, and Hu Orthotropic Ribbed Plate Method, abbreviated as SCH (Chui & Hu, 2002; Hu & Chui, 2004; Smith & Chui, 1988).

Fundamental frequency is examined because acceleration response, especially from resonance, depends greatly on fundamental frequency. Center deflection is examined because several design criteria are based on that quantity. Acceleration response from walking excitation is examined because predicted values can be compared directly to the ISO criterion.

The AISC design method is intended to be used to design steel framed floor systems and footbridges for vibration serviceability due to human activities. The ATC design method is intended to be used for design and retrofit of various floor structures to limit vibration to acceptable levels for humans. The ATC design method has provisions for both lightweight floor systems supported by wood joists, and heavy floor systems. In several instances, the ATC design method refers to a portion of the AISC design method for the design of heavy floor systems. The SCH design method is used to design wood floor systems for vibration serviceability. Specific modifications have been incorporated for use with engineered wood products, such as I-joists. This method created with designer-usability as a key goal.

5.2.1 Fundamental Frequency Estimation

In order to predict fundamental frequency, the ATC and AISC design guides use a single degree of freedom beam system model, which is based on previous work by Allen and Murray (Allen & Murray, 1993). The SCH method uses a single degree of freedom orthotropic place model, which is based on previous work by Smith and Chui (Smith & Chui, 1988).

Both the ATC and AISC Design guides define fundamental frequency as a function of deflection due to the uniformly distributed dead weight of the floor system. The expressions for fundamental frequency in both references differ because the ATC uses US Customary Units and the AISC uses SI units, but both guides calculate fundamental frequency (f_n) with expressions equivalent to the following:

$$f_n = \frac{\pi}{2}\sqrt{\frac{gEI_{eff}}{CwL^4}} \qquad 5\text{-}3$$

where:

g is gravitational acceleration;

EI_{eff} is the effective composite flexural stiffness in the primary direction, specifically defined by either ATC or AISC;

C is a constant to account for moment continuity over the end-supports, specifically defined by either ATC or AISC;

w is the actual weight of the floor, per unit length; and,

L is the span of the joists.

The two design guides provide different procedures for estimating both EI_{eff} and C. For this investigation C was assumed to be 1.0 because the floor joists are not continuous over the end-supports.

From the AISC procedure EI_{eff} was computed with the following expression:

$$I_{eff} = \frac{I_{comp}}{1 + 0.15\frac{I_{comp}}{I_{chords}}} \qquad 5\text{-}4$$

where:

I_{comp} is the fully composite moment of inertia of a single joist and subfloor t-beam; and,

I_{chords} is the moment of inertia of the joist chords alone.

Equation 5-4 is intended for use with simply-supported steel trusses to account for shear contributions to the deflected shape of the beam. For hot-rolled joists, I_{eff} is taken as I_{comp}, however, for cold-formed steel joists the web area is significantly smaller in ratio to the moment of inertia than for hot-rolled joists. Because shear stiffness is proportional to joist web area, the effective moment of inertia was used. Because there are no truss chords, the fully-effective moment of inertia of the joist cross-section was defined as I_{chords} when Eq. 5-4 was applied. For floor vibration design, the loads applied to the floor systems are light and do not result in local buckling within the cold-formed steel joist members. Gross section properties are assumed to be present. The term “effective” is used in floor vibration literature to indicate a reduction in some area within the cross-section to account for dynamic behavior. This is a different context than cold-formed steel design, where the term “effective” is used to describe section properties which are reduced due to local buckling effects.

The AISC procedure assumes that there is fully composite action in the cross-section for determining I_{eff}. The ATC procedure provides for both shear deflection and non-composite behavior within the cross-section with an expression containing several constants obtained from material testing of floor components. The composite stiffness, EI_{eff}, is defined by the following:

$$EI_{eff} = \frac{EI}{1 + \frac{\gamma EI}{C_{fn} EI_m}} \quad 5\text{-}5$$

where:

γ is the ratio of joist shear deflection to flexural deflection;

EI is the flexural stiffness of a single joist and subfloor t-beam; and,

EI_m is the flexural stiffness of the joist.

Tabulated values of γ for both wood and steel joist members are provided, and a recommended value of 0.0 is specified for cold-formed steel C-joists (Allen, Onysko, & Murray, 1999). This is the same value as recommended for hot-rolled joists with a length-to-depth ratio of greater than 12, and is not based on laboratory testing. Because cold-formed steel C-joists are similar to wood I-joists in terms of the ratio of web area-to-moment of inertia, the following expression for γ, recommended for wood I-joists, was computed:

$$\gamma = \frac{96EI_m}{0.4d \times 10^6 L^2} \quad 5\text{-}6$$

where:

d is the joist depth (in inches); and,

L is the joist span (in inches).

The values obtained range from 0.02 to 0.05, which suggests that shear deformation accounts for up to 5% of the overall deflection. The values for γ based on Eq. 5-6 were used for evaluating Eq. 5-5. The values for EI_m were based on the gross section properties, and provided by the joist manufacturer.

For light frame construction, the ATC procedure specifies an area reduction of the concrete and sheathing subfloor to account for behavior which is not fully-composite. The effective axial stiffness of the subfloor, EA_{top}, is computed with the following expression:

$$EA_{top} = \frac{EA_{flr}}{1 + \frac{10EA_{flr}}{S_{flr} {L_{flr}}^2}} \quad 5\text{-}7$$

where:

EA_{flr} is the combined axial stiffness of the subfloor assembly;

S_{flr} is the slip modulus for the floor deck to member connection; and,

L_{flr} is the width of the floor panels (48" for panelized floors and L for concrete topped floors).

Values of S_{flr} are provided for OSB and tongue-and-groove wood deck only; for both exclusively nailed, and nailed and glued connections. These values were obtained from materials testing. In order to use the procedure for FORTACRETE and UFS subfloors, assumed values were used. The slip modulus should not be greater than OSB for FORTACRETE and UFS subfloors because the fastener spacing is not reduced, so a value of 50000 lb/in/in was taken for all subfloor assemblies with FORTACRETE® and UFS, which corresponds to the value for nailed and glued OSB. Initially, the value for nailed OSB, without glue was assumed, but this resulted in unreasonably low predicted frequencies. For the OSB, which was also glued, the value of 50000 lb/in/in was used. The reduced effective axial stiffness is then used to calculate neutral axis location and moment of inertia (the EI in Eq. 5-5 of the composite T-beam section of one joist spacing).

The SCH Method is based on the dynamic characteristics of an orthotropic plate (Smith & Chui, 1988). A general deflection profile is determined from the stiffness properties of the floor system and the fundamental frequency is obtained from the solution to an energy method equation which equates potential energy (maximum deflection, zero velocity) to kinetic energy (zero deflection, maximum velocity. This method is relatively simple when used to calculate the fundamental frequency, but is requires the user to assume the mode shape of the floor. The original method has been modified to include the influence of several different structural components such as engineered wood joists, blocking, strapping, and ceilings. The following expression was used to compute fundamental frequency with the SCH method (Chui & Hu, 2002):

$$f_n = \frac{\pi}{2\sqrt{\rho}} \sqrt{D_x \left(\frac{1}{a}\right)^4 + 2D_{xy} \left(\frac{1}{ab}\right)^2 + D_y \left(\frac{1}{b}\right)^4} \qquad 5\text{-}8$$

where:

ρ is the superimposed dead weight of the floor system (kg/m^2);

D_x is the system flexural rigidity in the joist direction (Nm);

D_{xy} is the combined subfloor shear rigidity and joist torsional rigidity (Nm);

D_y is the system flexural rigidity in the transverse direction (Nm);

a is the span of the floor (m); and,

b is the width of the floor (m).

The three expressions for rigidity are evaluated based on the structural components of the floor system. The joist direction flexural rigidity is calculated in a similar fashion to the EI in Eq. 5-5, using the same area reduction from non-composite action expressed in Eq. 5-7. The values for S_{flr} are not the same as for the ATC method because they were determined from different experimental data, but they are of a similar order of magnitude. A value of 5x10^{6} N/m/m was taken for all subfloor assemblies with FORTACRETE and UFS, which corresponds to the value for a topped subfloor. For the OSB, which was nailed and glued, the value of 1x10^{8} N/m/m was used. The value for a topped subfloor is specifically given to be the same at that for a bare subfloor, partly justifying the assumed values of S_{flr} used in the ATC method. The expression for D_x is:

$$D_x = \frac{EI_{apparent}}{b_1} \tag{5-9}$$

where b_1 is the joist spacing.

The authors recommend incorporating the following method for calculating $EI_{apparent}$ to include the influence of shear deformation:

$$\frac{1}{EI_{apparent}} = \frac{1}{EI}\left(1 + \frac{48 f_s EI}{5GAa^2}\right) \tag{5-10}$$

where:

EI is composite moment of inertia of a single joist and subfloor T-beam, reduced for non-composite action using Eq. 5-7, with S_{flr} as specified in the SCH method;

f_s is the shear form factor;

G is the shear modulus (77000 MPa); and,

A is the joist cross-sectional area.

Equation 5-10 is intended for dimension lumber or wood I-joists, and f_s for cold-formed steel C-shaped joists is not provided. It also may be more appropriate to use the web area, rather than the entire cross-sectional area, because there is no distinction between web and cross-section area for dimension lumber. A generic expression for f_s was used, developed for finite element models of wood-based floors (Jiang, Hu, & Chui, 2004), and is given as:

$$f_s = \left[1 + \frac{3(D_2^2 - D_1^2)D_1}{2D_2^3}\left(\frac{t_2}{t_1} - 1\right)\right]\frac{4D_2^2}{10R^2} \tag{5-11}$$

where:

D_1 is the distance between the neutral axis and nearest surface of the flange;

D_2 is the distance between the neutral axis and extreme fiber;

t_1 is the web thickness;

t_2 is the flange width; and,

R is the radius of gyration of the cross-section.

The expression in Eq. 5-11 is intended for regular I-sections, but it was applied for cold-formed steel C-shaped joists because no appropriate values for f_s are given. When $EI_{apparent}$ was calculated for the laboratory and in situ floors, it the apparent value was found to be from 0.8% to 3.1% of the original composite value. It does not significantly impact the results discussed in this Section.

Unlike the AISC and ATC methods, the SCH method explicitly considers the transverse and shear behavior of the floor system. Subfloor flexural rigidity in the transverse direction, D_y, is:

$$D_y = \frac{\sum_{i=1}^{k} EI_b^i}{a} + \frac{EI_p b_1}{b_1 - t - \alpha^3 t} \qquad \text{5-12}$$

where:

EI_b^i is the flexural stiffness of transverse bracing member *i* (Nm^2);

k is the number of transverse bracing members (strong back, strapping, bridging, blocking);

EI_p is the non-composite flexural stiffness of the subfloor assembly (Nm);

t is the width of a joist;

α is h/H;

h is subfloor thickness; and,

H is thickness of subfloor and joist.

Specific methods, based on extensive testing, are provided for computing the bending stiffness of transverse bracing members (Chui & Hu, 2002). They are separated into members containing discrete elements, such as blocking and cross bridging, and continuous members, such as strongbacks, strapping and ceilings. The blocking and strapping row detail used in the experimental portion of this study did not have a blocking member at each joist spacing, which is not considered in the SCH method. Therefore, no contribution from blocking and strapping was considered in the calculation of D_y. The value of EI_b for the strongback was taken from gross section properties given by the manufacturer. The value of EI_b for the gypsum board ceiling was computer for laboratory and field configurations where a ceiling attached with resilient channel was present. The SCH method specifies specific material properties to be used in order to calculate EI_b for a ceiling attached to the joists with resilient channel, which is given in Eq. 5-13:

$$EI_b = \frac{E_{strap} I_s}{1 - \frac{Z^2}{\left(\beta + \frac{\pi^2}{S_2 L_2^2}\right) E_{strap} I_s}} \qquad 5\text{-}13$$

where:

E_{strap} is 9.5x10^9 N/m^2;

I_s is $\frac{b_s t_{cs}^3}{12}$;

b_s is 89 mm;

t_{cs} is the thickness of the ceiling;

Z is $\frac{d+t_{cs}}{2}$;

β is $\frac{1}{E_{strap} b_s t_{cs}} + \frac{Z^2}{E_{strap} I_s}$;

S_2 is 2x10^6 N/m/m; and,

L_2 Is the width of the floor (m).

For EI_p, there is no composite action assumed. The flexural stiffness of the subfloor and concrete topping for the transverse direction, and about their own neutral axes, are simply added together.

The combined subfloor shear rigidity and joist torsional rigidity is given by the following:

$$D_{xy} = \sum_{i=1}^{n} \frac{G_i t_i^3}{12} + \frac{C}{2b_1} \qquad 5\text{-}14$$

where:

n is the number material layers in the subfloor;

G_i is the shear modulus of the ith subfloor layer (N/m^2);

t_i is the thickness of the ith subfloor layer; and,

C is the joist torsional constant (Nm^2).

For concrete, a value of zero is suggested for G because the cracked slab is assumed to have no shear resistance. For the steel deck, G was taken as 77x10^9 N/m^2, which is the typical shear modulus of structural steel. For OSB, the SCH procedure suggests a G of 200x10^6 N/m^2, and for FORTACRETE a value of 1.52x10^9 N/m^2 was assumed, which is 20% of the elastic modulus. The value of C was supplied by the joist manufacturer.

In order to examine the predictive ability of the AISC, ATC, and SCH methods, comparisons were separately made for the laboratory test floors and in situ floor systems. The laboratory floors in the simple support reflect the idealization from the ATC and AISC models, which is a beam vibrating about simple supports. Because the SCH model uses a vibrating plate which is simply supported on all sides, the design width was assumed to be 3/2 times the test floor width to attempt to account for the unsupported edges. The value of 3/2 was selected because the effective floor width used by the AISC design guide for predicting acceleration response is a maximum of 2/3 of the actual width (Murray, Allen, & Ungar, 1997). Along with the ideal case, the prediction of fundamental frequency for the in situ floor systems was examined in order to check all three models against actual conditions.

The prediction of fundamental frequency in the ideal case is shown graphically in Figure 5-3, and the values are tabulated in Table 5-4 for each floor system. Figure 5-3 shows the estimated frequency plotted against the measured frequency, with a solid line indicating an exact match and dashed lines showing an error range of 10% and 20%. It is important to note that both the AISC and ATC methods are defined as being accurate for frequency values up to 15 Hz, but not limit is given in the SCH procedure (Allen, Onysko, & Murray, 1999). For this reason, the range plotted has been limited to 25 Hz, showing the trends in both the expected region of accuracy and outside it.

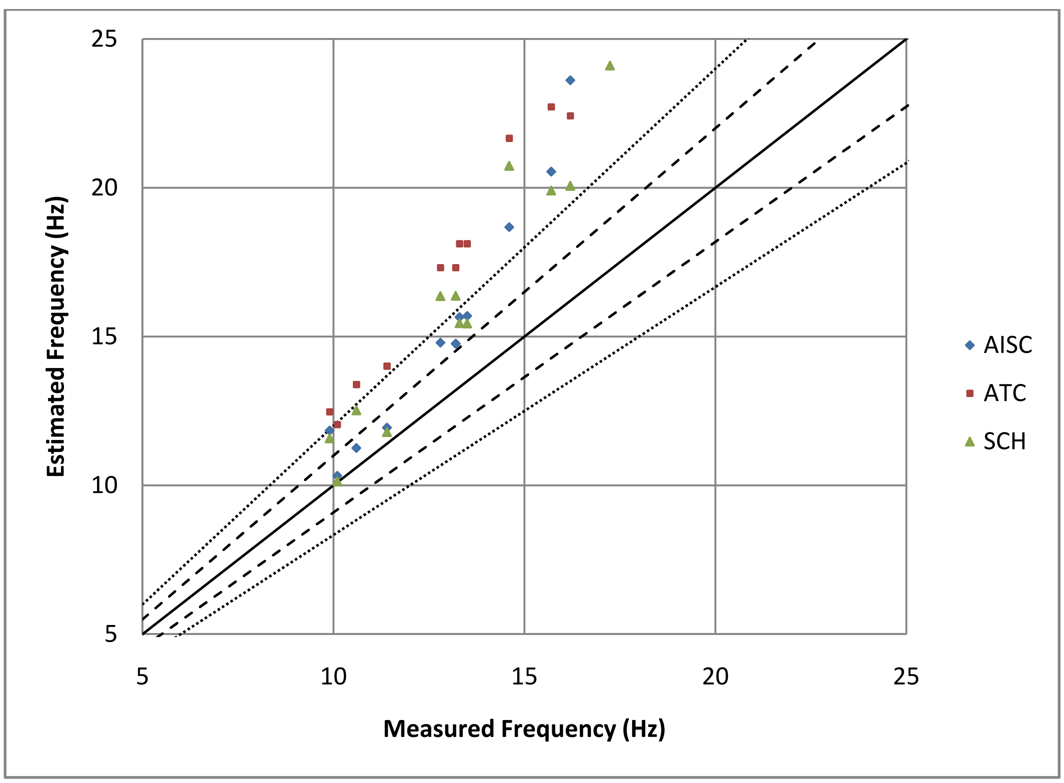

Figure 5-3: Fundamental Frequency Prediction (Laboratory, Simple Support Condition)

Two key observations can be made from Figure 5-3: all three methods tend to over-predict fundamental frequency; and, the accuracy of all three methods decreases with increasing fundamental frequency, most significantly after the estimated frequency exceeds approximately 13 Hz. The relevance to a designer is that the method appears to be unconservative by over-predicting fundamental frequency, and the method is more accurate approaching the more sensitive range of frequencies for occupant comfort and walking excitation. Both the AISC and SCH methods are accurate to within 20% of the measured values for several of the lower frequencies. The ATC did not prove to be accurate within 10% for any of the floors examined, while it is shown in Table 5-4 that both the AISC and SCH methods are in some cases.

Table 5-4: Fundamental Frequency Prediction (Laboratory, Simple Support Condition)

Floor Name	f_1 Lab	f_1 AISC	% error AISC	f_1 ATC	% error ATC	f_1 SCH	% error SCH
LF14.5A	19.1	30.1	58%	32.5	70%	26.5	39%
LF14.5B	17.2	27.8	61%	27.1	57%	24.1	40%
LF14.5C	17.7	30.1	70%	32.5	83%	26.5	50%
LF14.5D	16.2	23.6	46%	22.4	38%	20.1	24%
LF14.5E	15.7	20.5	31%	22.7	45%	19.9	27%
LF14.5F	14.6	18.7	28%	21.7	48%	20.7	42%
LF17.0A	13.5	15.7	16%	18.1	34%	15.4	14%
LF17.0B	13.3	15.7	18%	18.1	36%	15.4	16%
LF17.0C	12.8	14.8	16%	17.3	35%	16.4	28%
LF17.0D	13.2	14.8	12%	17.3	31%	16.4	24%
LF19.5A	11.4	11.9	5%	14.0	23%	11.8	3%
LF19.5A$_{iv}$	10.1	10.3	2%	12.0	19%	10.1	0%
LF19.5B	10.6	11.3	6%	13.4	26%	12.5	18%
LF21.8A	9.9	11.8	20%	12.5	26%	11.6	17%

The prediction of fundamental frequency for actual constructed floors is shown graphically in Figure 5-4, and the values are tabulated in Table 5-5 for each floor system. In Figure 5-4, the estimated frequency is plotted against the measured frequency, with a solid line indicating an exact match and dashed lines showing an error range of 10% and 20%. As stated above, both the AISC and ATC methods are accurate for values up to 15 Hz.

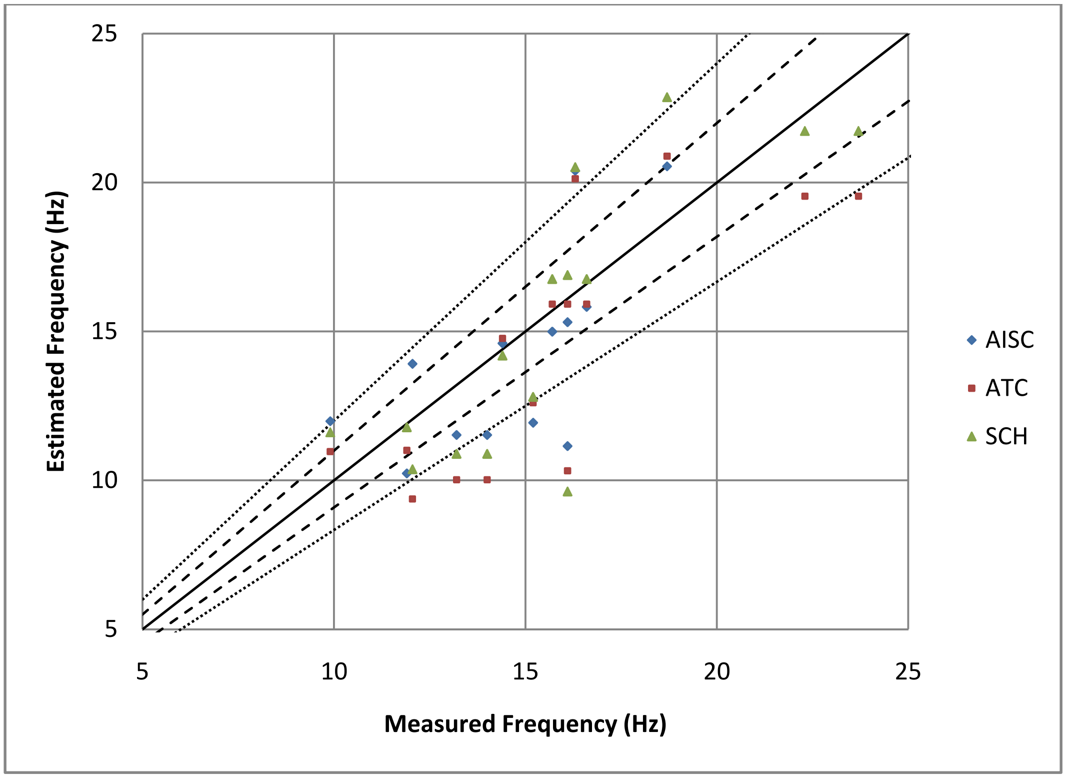

Figure 5-4: Fundamental Frequency Prediction (In Situ, Balloon Framing Condition)

It can be seen in Figure 5-4 that there is a wider spread of predicted values, and that fundamental frequency is both over and under-predicted. However, as the measured values approach the critical frequency of approximately 8 Hz, the tendency is for the predicted frequencies to be lower than the measured values, which is conservative from a design perspective. It is shown in Table 5-5 that all of the three methods were both extremely accurate and inaccurate for different floor systems, irrespective of which floor system is examined. Because the errors are both positive and negative, it is meaningful to examine the standard deviation in order to determine the most accurate method. The ATC method has the lowest standard deviation of error, and can therefore be assumed to be the most accurate of the three.

Table 5-5: Fundamental Frequency Prediction (In Situ, Balloon Framing Condition)

Floor Name	f_1 Field	f_1 AISC	% error AISC	f_1 ATC	% error ATC	f_1 SCH	% error SCH
CG601	14.4	14.6	1%	14.8	3%	14.2	-1%
CG604	16.3	20.4	25%	20.1	23%	20.5	26%
CG805	15.2	11.9	-21%	12.6	-17%	12.8	-16%
CGMH6	15.7	15.0	-5%	15.9	1%	16.8	7%
CGMH7	16.6	15.8	-5%	15.9	-4%	16.8	1%
CW707	16.1	15.3	-5%	15.9	-1%	16.9	5%

CW708	18.7	20.5	10%	20.9	12%	22.9	22%
CW709	9.9	12.0	21%	11.0	11%	11.6	17%
CW805	11.9	10.2	-14%	11.0	-8%	11.8	-1%
OK401	22.3	27.1	22%	19.5	-12%	21.7	-3%
OK402	23.7	27.1	14%	19.5	-18%	21.7	-8%
OK403	32.8	47.0	43%	32.1	-2%	37.2	13%
DDG1	12.0	13.9	15%	9.4	-22%	10.4	-14%
DDG2	14.0	11.5	-18%	10.0	-28%	10.9	-22%
DDG3	13.2	11.5	-13%	10.0	-24%	10.9	-17%
DDG4	16.1	11.1	-31%	10.3	-36%	9.6	-40%
		Std. Dev.	**20%**		**16%**		**17%**

5.2.2 Static Deflection from 1 kN Point Load Estimation

The AISC design method includes a procedure for calculating static deflection which does not correspond to Onysko's criterion, so it is not examined. The ATC design method assumes that the floor system behaves as a one-way floor panel, the width of which is determined by a calculated number of effective joists (Allen, Onysko, & Murray, 1999). Center deflection due to a 1 kN point load at mid-span (Δ_p) is given by the following expression:

$$\Delta_p = \frac{C_{pd}}{N_{eff}} \frac{PL^3}{48EI_{eff}} \qquad \text{5-15}$$

where:

C_{pd} is a continuity factor for point loading;

N_{eff} is the number of effective joists;

P is 225 lb (1 kN);

L Is the span length (in); and,

EI_{eff} Is the effective flexural stiffness from Eq. 5-5, with C_{fn} replaced by C_{pd}.

Similar to the assumption made for fundamental frequency, C_{pd} is assumed to be 1.0 because the joists do not extend beyond one bay. Because of this, EI_{eff} is the same for calculating Δ_p as it is for calculating f_n. The procedure for calculating N_{eff} is contains empirical coefficients which were obtained from testing floors supported by wood joists. N_{eff} is governed by the ratio of primary stiffness of the joist panel to the total stiffness in both primary and transverse directions (Allen, Onysko, & Murray, 1999). The following considerations and assumptions were made in order to calculate N_{eff}:

- the strongback detail was considered as a transverse flexural element;

- the blocking and strapping details were not considered;
- the section properties of LEVELROCK topping were based on a rectangular section with a depth equal to the average thickness in the transverse direction;
- the contribution from the ceiling was not considered; and,
- the only component with a shear contribution was the strongback.

These assumptions were made in order to use the procedure as outlined. It is clear that the shear stiffness of the subfloor must be considered for this method to be more accurate.

The SCH method uses a solution using the ribbed plate theory proposed by Timoshenko and Woinowsky-Krieger in 1959 to determine static deflection from a point load at the geometric center of an orthotropic ribbed plate with simply supported edges (Chui & Hu, 2002). For comparison against Onysko's criterion, center deflection due to a 1 kN point load at mid-span (Δ_p), in meters, is given by the following expression:

$$\Delta_p = \frac{4P}{ab\pi^4} \sum_{m=1.3.5\ldots}^{17} \sum_{n=1,3,5\ldots}^{35} \frac{1}{\left(\frac{m}{a}\right)^4 D_x + 2\left(\frac{mn}{ab}\right)^2 D_{xy} + \left(\frac{n}{b}\right)^4 D_y} \qquad \text{5-16}$$

where P is 1 kN. All other variables are described previously. The ranges of the summations are suggested by the authors to ensure a convergent solution.

The prediction of static deflection for laboratory floor systems, in the simple support condition, is shown graphically in Figure 5-5, and the values are tabulated in Table 5-6 for laboratory and in situ floor systems. Figure 5-5 contains estimated frequency plotted against the measured frequency, with a solid line indicating an exact match and dashed lines showing an error range of 10% and 20%. Values from the laboratory floor systems in the simple support condition are examined because that condition is the best representation of the models used by the ATC and SCH. In Table 5-6, the in situ data is also included, which is from the balloon framing condition. The in situ data should not be given much weight in terms of assessing the accuracy of the predictions because there is a high degree of error associated with the measurement of deflection in the field.

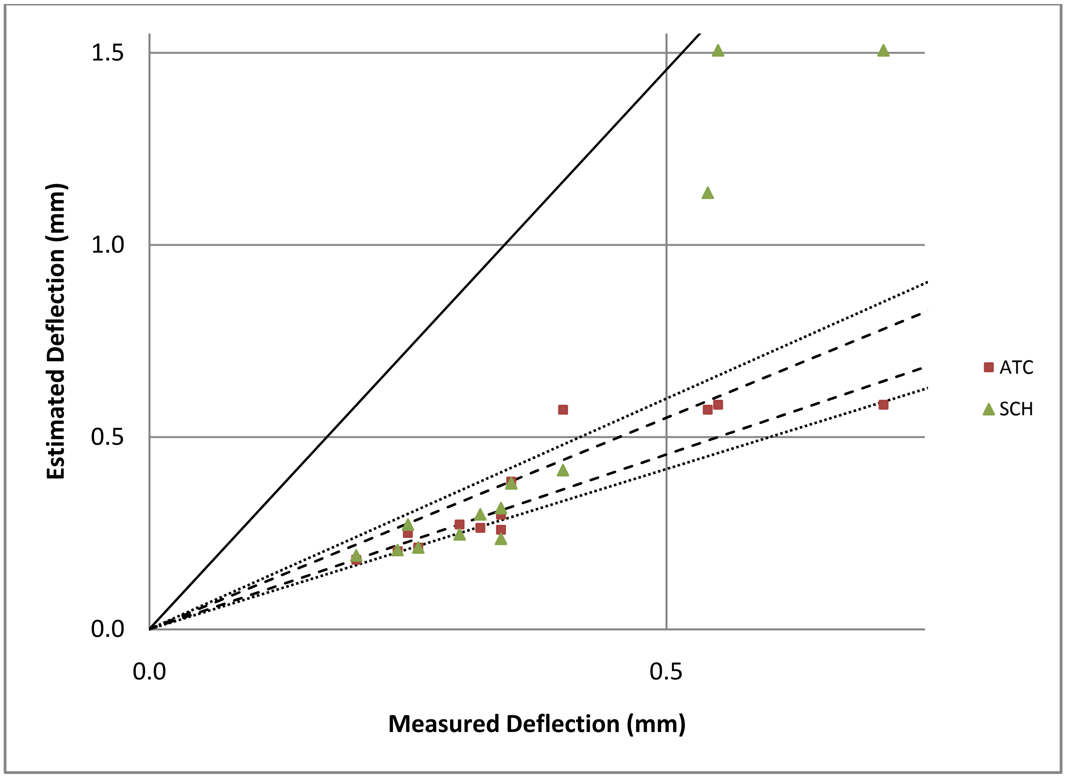

Figure 5-5: Center Deflection Prediction (Laboratory, Simple Support Condition)

It can be seen in Figure 5-5 that the majority of estimated values are spread within 20% on both sides of the measured value. The SCH method appears to under-predict deflection by a larger magnitude and in more cases than the ATC method, which is unconservative from a design perspective. THE SCH method exhibited a significant overestimation for three floor systems. From Table 5-6, it can be seen that the ATC overestimated the deflection of LF14.5D by 43%, which is has a low aspect ratio, and a FORTACRETE subfloor without topping, but with a ceiling in place. The SCH method was within 3% for this floor. This suggests that the ceiling plays a significant role in the deflection behavior of a floor system without topping: the ceiling was accounted for in the SCH method, but not the ATC method. The SCH method had an error of 110% or greater for the floor systems with no topping or ceiling: LF14.5A, LF14.5B, and LF14.5C. This indicates that the blocking and strapping requires consideration when the subfloor assembly, including ceiling, is relatively flexible. For floors with a topping, the general trend was that the measured deflection was underestimated by approximately 10% to 20%.

Table 5-6: Center Deflection Prediction (In Situ and Laboratory, Simple Support Condition)

Floor Name	Δ_p Measured	N_{eff} ATC	Δ_p ATC	% error ATC	Δ_p SCH	% error SCH
CG601	0.46	2.08	0.31	-32%	0.37	-21%
CG604	0.28	1.81	0.23	-18%	0.25	-11%
CGMH6	0.36	2.01	0.29	-19%	0.23	-35%
DDG1	0.39	1.85	0.30	-22%	0.33	-17%
DDG2	0.37	3.37	0.32	-15%	0.27	-28%
DDG3	0.20	3.37	0.32	58%	0.27	34%
LF14.5A	0.55	1.57	0.58	6%	1.51	174%
LF14.5B	0.54	1.51	0.57	6%	1.14	110%
LF14.5C	0.71	1.57	0.58	-18%	1.51	112%
LF14.5D	0.40	1.51	0.57	43%	0.41	3%
LF14.5E	0.25	1.84	0.25	0%	0.27	9%
LF14.5F	0.20	2.53	0.18	-10%	0.19	-4%
LF17.0A	0.34	2.03	0.30	-12%	0.32	-7%
LF17.0B	0.32	2.29	0.26	-18%	0.30	-7%
LF17.0C	0.26	2.81	0.21	-18%	0.21	-18%
LF17.0D	0.24	2.95	0.20	-15%	0.21	-14%
LF19.5A	0.35	2.29	0.38	10%	0.38	8%
LF19.5Aiv	0.35	2.29	0.38	10%	0.38	8%
LF19.5B	0.30	3.20	0.27	-9%	0.25	-18%

5.2.3 RMS Acceleration Response from Walking Excitation Estimation

The AISC and ATC methods contain an identical procedure for calculating the resonant RMS acceleration response due to walking excitation. The procedure, as described by the AISC, will be examined and compared to the measured values. The method is based on work by Allen and Murray (1993). The acceleration limits are from the ISO 2631-2 (1989) baseline curve, with multipliers of 10 for offices or residences, 30 for malls, and 100 for outdoor footbridges. For design purposes, using 0.8 to 1.5 times the recommended limit is suggested, depending on duration and frequency of vibration events (Murray, Allen, & Ungar, 1997). This will be discussed more in the following Section.

The peak acceleration due to walking $\left(\frac{a_p}{g}\right)$ at occurs at resonance, where $f_n = f_{step}$, and is expressed as:

$$\frac{a_p}{g} = \frac{R\alpha_i P}{\beta W} \qquad 5\text{-}17$$

where:

R is a reduction factor;

α_i is the dynamic coefficient of the ith harmonic;

P is the design weight for walking, assumed to be 0.7kN (157 Lbs);

β is the assumed modal damping ratio; and,

W is the effective weight of the floor.

The procedure uses only one harmonic component, which is assumed to be at a frequency corresponding to the fundamental frequency of the single degree of freedom idealized floor system. The floor is assumed to be at resonance, which will dominate all other responses. The reduction factor is added because steady-state resonant response does not occur from walking excitation, and the annoyed person is typically some distance from the source of walking excitation. A recommended value of R is 0.5 for floor systems with two-way behavior (Murray, Allen, & Ungar, 1997).

When the empirical expression for α_i, and the step frequency is set to the fundamental frequency, the RMS acceleration response is expressed as:

$$\frac{a_p}{g} = \frac{P_o e^{(-0.35 f_n)}}{\beta W} \quad \text{5-18}$$

where P_o is a constant force of 0.29 kN (65 lb) for floor systems (Allen, Onysko, & Murray, 1999).

The design damping ratio is assumed, and various sources recommend values ranging from 0.02 (Murray, Allen, & Ungar, 1997) to 0.12 (Allen, Onysko, & Murray, 1999). The effective weight is determined by applying the dead weight of the floor system to an area defined by the joist span and effective joist panel width. The effective panel width is defined as:

$$B = C_j \left(\frac{D_s}{\frac{I_{eff}}{S}} \right)^{0.25} L \quad \text{5-19}$$

where:

C_j is 2.0 for typical floor systems; and,

D_s is the transformed subfloor moment of inertia, per unit length (Allen, Onysko, & Murray, 1999).

In order to examine the prediction of the RMS acceleration response in the in situ floors, the measured response was compared against two cases: the estimated response from Eq. 5-18, using experimental data for f_n and β; and, the estimated response from Eq. 5-18, using calculated data following the AISC procedure for f_n and β. A value of 0.03 was taken for because there were typically no full-height partitions present, based on the AISC recommendation. Both estimates are plotted against the measured value in Figure 5-6, and the absolute values are tabulated in Table 5-7.

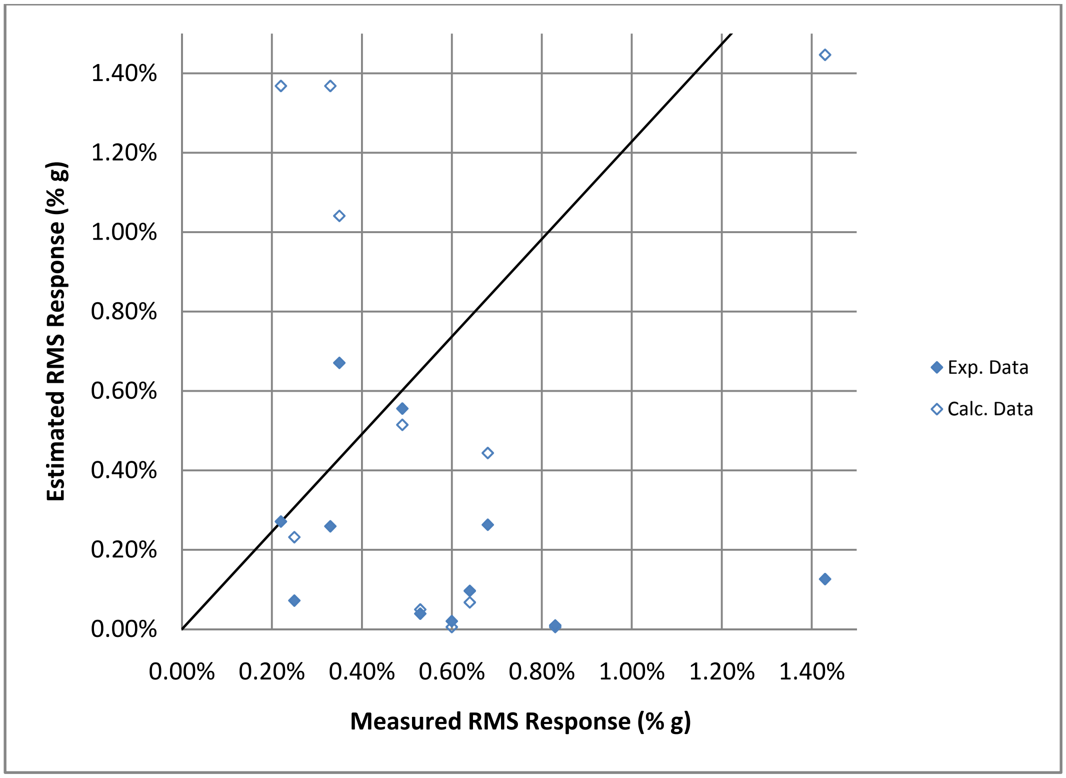

Figure 5-6: Comparison of RMS Response Estimates for In Situ Floors

It is expected that the estimated value of RMS response will be greater than the measured value because the estimate is based on the assumption that the floor is being excited in a resonant manner. Typically, this was not the case, as can be seen in Figure 5-6. Potentially, this discrepancy could be explained because the excitations were provided by an individual weighing 0.8 kN (180 lb). Other possible explanations are that for floors with closely spaced natural frequencies, there is a significant contribution from the second mode of response, or that damping was underestimated, or measured incorrectly. From a designer's perspective, the general trend showing that the acceleration calculated using the design procedure is less than the actual measured RMS response is unconservative. However, it has been shown to be extremely over-conservative in some cases, and may lead to overdesign if it is used exclusively. It can also be concluded that predictions of acceleration using this method are typically very inaccurate.

Table 5-7: Comparison of RMS Response Estimates for In Situ Floors

Floor Name	a_p/g Measured	a_p/g using Exp. Data	a_p/g using Calc. Data
CG601	0.680%	0.263%	0.444%
CG604	0.640%	0.097%	0.068%
CW707	0.250%	0.072%	0.232%
CW708	0.530%	0.039%	0.050%
CW709	0.490%	0.556%	0.515%
DDG1	0.350%	0.671%	1.041%
DDG2	0.330%	0.259%	1.368%
DDG3	0.220%	0.271%	1.368%
DDG4	1.430%	0.126%	1.447%
OK401	0.600%	0.020%	0.006%
OK402	0.830%	0.009%	0.006%

5.3 Recommended Design Procedure and Suggestions for Modification

5.3.1 Design Procedure for Floors Supported by Cold-Formed Steel Joists Using Current Methods

Based on the previous Section, it is clear that none of the design models evaluated prove more accurate than the other for predicting fundamental frequency, center deflection and RMS acceleration response of floors supported with cold-formed steel joists. This is because the AISC model was developed for heavy floor systems with high transverse stiffness, and both the ATC and SCH methods rely on several empirical coefficients that have been obtained from wood construction. This is clearly illustrated in Figure 5-5, by noting that the prediction of center deflection is least accurate when contributions from the ceiling and blocking and strapping details are observed experimentally, but not adequately accounted for through calculation.

For the prediction of fundamental frequency, the ATC procedure is recommended for the following reasons:

- compared to the AISC procedure, the predicted values provided more conservatism approaching the critical range, and more accurate for the field floors based on standard deviation of error;
- compared to the AISC procedure, shear deflection and non-composite action are specifically addressed;
- compared to the SCH procedure, the method is simpler and easier to understand; and,

- the ATC procedure can also be used to compute center deflection from a 1 kN point load.

For the prediction of center deflection from a 1 kN point load, the ATC procedure is recommended for the following reasons:

- compared to the AISC procedure for calculating N_{eff}, the ATC procedure is conceptually appropriate for lightweight floor systems (the AISC procedure has not been discussed);
- compared to the SCH procedure, the ATC procedure is slightly less unconservative, and less prone to extremely over-conservative predictions; and,
- compared to the SCH procedure, the ATC procedure is more accurate for floor systems without a topping and ceiling, which is relevant in single family residential construction.

The following procedure is recommended for to be used for the design of floor systems supported by cold-formed steel joists, with or without a non-structural concrete topping, and intended for residential occupancy:

1. Determine fundamental frequency using the method described by the ATC Design Guide 1. If it is greater than 15 Hz, skip Step 2.
2. If the fundamental frequency is below 15 Hz, check the RMS acceleration response from walking excitation, using the method described by the ATC Design Guide 1. The response should be less than the value prescribed for residential occupancy, which is 0.5% g. A slight modification to this is described in the following Section.
3. Check the static deflection from a 1 kN point load against Onysko's limit, as described by the ATC Design Guide 1.
4. Revise floor system design if criteria for Step 2 or 3 are not met.

If the fundamental frequency is greater than 15 Hz then the RMS response from walking does not need to be checked for the following reasons: the ATC method is not accurate for values above 15 Hz (Allen, Onysko, & Murray, 1999); it is conceptually incorrect to use a resonance-based model to predict acceleration response from walking above approximately 8.8 Hz, the values of the dynamic coefficients for higher forcing frequencies are extrapolated from test data (Murray, Allen, & Ungar, 1997); and, it has been shown in previous research that a minimum fundamental frequency of 15 Hz is an adequate design criterion for lightweight floor systems (Johnson, 1994).

If the fundamental frequency is estimated to be below 8 Hz, it may be beneficial to consider altering the design so that it is increased to greater than 10 Hz. This is not necessary, provided Step 2 above is satisfied, but will ensure that the floor will not have a resonant response under most occupant-induced excitations. When considering options to increase fundamental frequency at the design stage, increasing joist moment of inertia is the recommended step. This will increase primary stiffness with a small impact on mass, as well as give a quantifiable indication to the designer of the increase. Because transverse stiffening measures are not adequately accounted for with the floor system model, changes in fundamental frequency may not be determined with accuracy. However, when transverse stiffening details, including blocking and strapping, strongback, and ceiling fixed with resilient channel are used, the estimated fundamental frequency will be slightly conservative.

Based on the results from the in situ testing conducted in this study, it can be assumed that the predicted RMS acceleration response will be conservative. This is reinforced because the actual end-framing conditions and transverse stiffening elements have been shown to have more of an influence on the response than is predicted.

Static deflection must be checked for all floor systems, regardless of fundamental frequency. Both the ATC and AISC design guides recommend checking static deflection if the fundamental frequency is greater than 10 Hz to 15 Hz. Floors without topping are more susceptible to local defection problems, and tend to have high fundamental frequencies. DDG1, which had a bare FORTACRETE subfloor, had a fundamental frequency of only 12.0 Hz because of the span length.

5.3.2 Proposed Modifications to Current Methods Specifically for Floors Supported by Cold-Formed Steel Joists

Based on the testing component of this study, design damping ratios should be based on the values presented in Table 4-9, as they are based on the specific joist and subfloor materials, and construction techniques to cold-formed steel floor systems. These values should result in more accurate predictions of acceleration response.

Both the AISC and ATC methods recommend a single limiting acceleration value to be applied to the result of Eq. 5-18. Because human sensitivity to vibration varies with frequency, this limit can be modified to incorporate the fundamental frequency of the floor system, by applying the frequency weighting method shown in Eq. 5-1. This will reduce the design requirements in frequency ranges where human sensitivity is also reduced. Appendix D contains a design example based on the recommended procedure from the previous Section, including the above modifications.

Assumptions for the variables and coefficients for use in design of floor systems supported by cold-formed steel joists can be modeled after the assumptions described previously. However, significant improvement in the accuracy of fundamental frequency and center deflection predicted using the ATC procedure can be achieved if further research is directed towards obtaining values corresponding to the construction materials and details specific to floor systems supported by cold-formed steel joists.

Testing and finite element modeling could be used to determine the following:

- an appropriate shear form factor (γ) for cold-formed steel joists to be used in calculating the effective primary bending stiffness;
- an appropriate slip modulus (S_{flr}) for OSB, FORTACRETE®, and UFS subfloors connected to cold-formed steel joists with self-tapping screws;
- a method for calculating the number of effective joists (N_{eff}) which incorporates the shear and bending contribution from the blocking and strapping detail (as tested) and ceiling fixed with resilient channels; and,
- replacing the dynamic continuity factor (C_{fn}) with an updated model which considers partial end restraint and transverse stiffness for the calculation of fundamental frequency.

Previous research at the University of Waterloo has examined incorporating a semi-rigid connection in the single degree of freedom beam model used to predict fundamental frequency using the ATC method (Tangorra, 2005). This research did not consider the balloon end-framing condition. That investigation could be continued by examining the test results from this study. Also, the contribution of transverse properties on the fundamental frequency could be incorporated into the ATC model. The fundamental frequency could be calculated based on the deflection of a simply supported beam with moment springs at the support, and a linear spring at mid-span. This moment springs at the supports would account for the rotational restraint imposed by the real end-support conditions, and the linear spring would account for transverse behavior of the floor system.

Chapter 6. Conclusions and Recommendations

Analysis of the laboratory and in situ test data has provided conclusions on the effect of several construction details on the transverse stiffness properties of a floor system, and a recommended design damping ratio for vibration serviceability calculations. It was found that the subfloor, ceiling, and strongback details all have an observable effect on static deflection and relative frequency separation. The UFS subfloor provided more transverse stiffness than a topped LEVELROCK assembly. A ceiling attached with resilient channel was also found to increase the transverse stiffness of a floors system, when a strongback detail was not present. The effectively restrained strongback detail provided the greatest increase in transverse stiffness out of all the details that were compared. However, when a ceiling and a strongback were combined, this effect was not observed. The logarithmic-decrement method damping ratio for the in situ floors was re-analyzed using a filtering technique to isolate the fundamental mode of vibration in the time domain. Recommended damping ratios for design of floors supported by cold-formed steel joists, with consideration for various construction details, are given in Table 4-9.

In order to design a floor system to meet occupant requirements for vibration serviceability, designers must be able to compare a predicted characteristic or response against established serviceability criteria. Fundamental frequency, static deflection due to a 1 kN point load, and acceleration response from walking excitation are values which are commonly considered by vibration serviceability criteria. Current design methods for floor systems based on a cold-formed steel framework, or containing FORTACRETE or LEVELROCK, do not exist. For this reason, existing design methods published by the AISC, ATC, and SCH design methods used to predict fundamental frequency, static deflection, and acceleration. These estimated values were compared against the measured values from the vibration testing component of this study, and several observations were made. Material and cross-section properties used for design calculations are given in Appendix C.

All three methods over-predict the fundamental frequency for the laboratory floors, and their accuracy breaks down noticeably above 15 Hz. Based on the prediction of fundamental frequency in the in situ floors, the ATC method is the best option for designers. This method has the advantages of being conservative in the critical frequency range, and explicitly considers shear deformation and non-composite behavior. It is conceptually more applicable than the AISC method, and simpler to use than the SCH method. The inaccuracy above 15 Hz is not critical because dynamic excitation from residential and commercial used is typically walking excitation, which has a highest harmonic of 8 Hz.

The SCH and ATC methods are both slightly unconservative and inaccurate when used to predict static deflection. However, the ATC procedure is more accurate for floor systems without a topping and ceiling, which is relevant in single occupancy residential construction. Based on the prediction of static deflection, the ATC method, in its current state, is the best option for designers. This recommendation has the added benefit of requiring one design method for calculating static and dynamic response, reducing the number of design calculations required.

The single degree of freedom beam resonant model for walking excitation was used to predict the acceleration response using both predicted and measured values for fundamental frequency and damping. This model is common to the AISC and ATC methods. The predicted response is generally inaccurate and the trend is that it is less than the measured response, which is unconservative. Analysis of a larger data set is required before the accuracy of this model can be evaluated; however, this illustrates the need for designers to be accurate in their estimation of floor properties.

6.1 Recommended Design Procedure

A recommended design procedure, using currently published design methods is provided. This method should provide the best approach for designers to ensure floor systems meet the ISO Acceleration limit, and Onysko's static stiffness requirements. The method specifically addresses resonant response due to walking excitation, and localized deflection due to footfalls.

The recommended design procedure is as follows:

1. Determine fundamental frequency using the method described by the ATC Design Guide 1. If it is greater than 15 Hz, skip Step 2.
2. If the fundamental frequency is below 15 Hz, check the RMS acceleration response from walking excitation, using the method described by the ATC Design Guide 1. The response should be less than the value prescribed for residential occupancy, which is 0.5% g. A slight modification to this is described in the following Section.
3. Check the static deflection from a 1 kN point load against Onysko's limit, as described by the ATC Design Guide 1.
4. Revise floor system design if criteria for Step 2 or 3 are not met.

It should be cautioned that the effectiveness of this procedure is based on the ability for the acceptability criteria to actually provide satisfactory responses for occupant comfort. In order to reduce the possibility of an unsatisfactory floor design, both the ISO and Onysko's criteria are applied.

However, perception of vibration and occupant comfort are very subjective, and this procedure relies on the requirements set by the acceptability criteria as being sufficient.

6.2 Recommended Modifications to Current Design Methods

Several recommendations for modifying the ATC design method were made so that it will be more accurate and applicable to the materials used for cold-formed steel floor systems. The following modifications can be made to the calculation procedure:

- use design damping ratios presented in Table 4-9 when calculating dynamic response; and,
- reduce limiting acceleration based on ISO limiting curve when fundamental frequency is above 8 Hz using Eq. 5-1.

The following values should be determined from further testing and analysis:

- an appropriate shear form factor (γ);
- an appropriate slip modulus (S_{flr}) for OSB, FORTACRETE, and UFS subfloors;
- an improved method for calculating the number of effective joists (N_{eff}); and,
- an updated model which considers partial end restraint and transverse stiffness for the calculation of fundamental frequency.

6.3 Recommended Modifications to Floor Testing Program

The testing methods used for this study were based, for the most part, on existing test procures developed during the previous studies conducted at the University of Waterloo. The measurement and data acquisition equipment were inherited from the previous testing as well. A review of several publications (Hanagan, Raebel, & Trethewey, 2003; Xin, 1996) has provided the following recommendations for updating the test procedures and equipment used in future testing at the University of Waterloo.

The following modifications are recommended:

- Replace existing software-based analysis and data acquisition system with a dedicated signal analyzer. This will increase reliability, speed of analysis, and improve portability for in situ testing (Hanagan, Raebel, & Trethewey, 2003).
- Ensure replacement analyzer has built-in signal conditioning to reduce the amount of equipment required (Hanagan, Raebel, & Trethewey, 2003).

- Increase the number of channels, cables, and accelerometers available for testing. Three accelerometers is sufficient for measuring lower-order frequencies, but in order to obtain higher-order frequencies and mode shapes, as many as 300 test points may be required (Hanagan, Raebel, & Trethewey, 2003). Clearly, using 300 channels is cost-prohibitive, but more channels available will reduce the number of test repetitions required per floor (Xin, 1996).
- Replace existing accelerometers with higher-sensitivity models, a minimum of 1 V/g is suggested. The current models have a sensitivity of 0.1 V/g, and capture a large amount of noise when the signal is amplified (Hanagan, Raebel, & Trethewey, 2003). The better accelerometers will significantly improve the measurements required for determining the damping ratio.
- Perform the heel drop on a force-measuring plate on the floor system. Measurement of the actual force will provide a high quality frequency response function for correlating force to acceleration (Hanagan, Raebel, & Trethewey, 2003).
- Apply excitation force at a location other than the center of the floor. This will avoid nodal lines for the lower-order modes, and provide better results (Hanagan, Raebel, & Trethewey, 2003).

6.4 Recommended Areas of Future Study

Apart from the additional testing and analysis recommended in Section 6.2, any additional in situ floor testing will be beneficial because it will expand the database of test results for in-service floors. The conclusions in this thesis, and those made by Davis (2008) will benefit from the support of a larger data set. This is especially relevant for determining an appropriate design damping ratio, as the values in Table 4-9 were based on one or two floors in some cases. A database for the results of all the past, current, and future testing done at the University of Waterloo should be created.

A relevant research topic for floor systems supported by cold-formed steel joists is an in-depth study of a way of predicting the number of effective joists that participate in the static and dynamic response of the floor. The existing empirical formulas could be modified, or a new model developed based on the cold-formed steel floor structural framework.

Finally, additional research in the field of occupant sensitivity should be performed. One recommended study is an occupant satisfaction survey of the floor systems tested in the in situ testing component of this study. Because the individual floors were repeated throughout the story and over several levels, a large set of responses would be available. Having stood on all of the laboratory and in situ floor systems himself, it is the author's opinion that the perception of vibration is heightened by participants in a floor vibration study, especially in a laboratory setting. Future studies on occupant sensitivity and acceptability should be conducted in a manner so that the participants are distracted and unaware that they are being exposed to vibrations prior to their assessment of the floor.

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Appendix A – Dietrich Design Group Test Floor Drawings

DRAWING INDEX

DRAWING NUMBER	DESCRIPTION	REV DATE
CF1	INDEX / APPROVAL SHEET	11.30.05
CF1a	PLATFORM LAYOUT	11.30.05
CF1b	FLOOR FRAMING	11.30.05
CF1c	ROOF FRAMING	11.30.05
CF2	CFS/PLYWOOD FULL SECTIONS	1.18.05
CF3	CFS/SCP FULL SECTIONS	1.18.05
CF4	CFS/LEVELROCK W/SOUND MAT	1.18.05
CF5	CFS/LEVELROCK FULL SECTIONS	1.18.05
CF6	CFS/POURED-IN-PLACE CONCRETE	1.18.05
CF7	CFS/PRE-CAST CONCRETE	1.18.05
CF8	CFS/HAMBRO AT FLOOR	11.30.05
CF9	CFS/EP AT FLOOR	11.30.05
CF10	TYPICAL DETAILS	1.18.05
CF11	CFS/PLYWOOD AT FLOOR	1.18.05
CF11A	CFS/PLYWOOD AT FLOOR	1.18.05
CF12	CFS/SCP AT FLOOR	1.18.05
CF12A	CFS/SCP AT FLOOR	1.18.05
CF13	CFS/LEVELROCK W/SOUND MAT AT FLOOR	1.18.05
CF13A	CFS/LEVELROCK W/SOUND MAT AT FLOOR	1.18.05
CF14	CFS/LEVELROCK AT FLOOR	1.18.05
CF14A	CFS/LEVELROCK AT FLOOR	1.18.05
CF15	CFS/POURED-IN-PLACE CONCRETE AT FLOOR	1.18.05
CF15A	CFS/POURED-IN-PLACE CONCRETE AT FLOOR	1.18.05
CF16	CFS/HOLLOW-CORE PRE-CAST CONCRETE AT FLOOR	11.30.05
CF16A	CFS/HOLLOW-CORE PRE-CAST CONCRETE AT FLOOR	11.30.05
CF17	CFS/HAMBRO AT FLOOR	11.30.05
CF17A	CFS/HAMBRO AT FLOOR	11.30.05
CF18	CFS/SOLID PRE-CAST PLANK AT FLOOR	11.30.05
CF19	TRUSS PROFILE AND GABLE SECTION	1.19.05
CF20	SHEARWALL/OPENING FRAMING ELEVATION	1.19.05
CF21	SHEARWALL/OPENING FRAMING ELEVATION	1.19.05
CF22	DETAILS	1.19.05
CF23	DETAILS	1.19.05
CF24	DETAILS	1.19.05
CF26	GABLE OUTRIGGER PANEL SHOP DRAWING	1.19.05

COLD-FORMED STEEL DESIGN FOR:

SOUND TEST CHAMBER

HAMMOND, IN

NOTES:
1. PROJECT IS FOR THE TEMPORARY TESTING OF FLOOR ASSEMBLIES.
2. COORDINATE ELECTRICAL ROUGH-IN WITH BERT PASSALAQUA PRIOR TO CLOSING WALLS AND CEILINGS.
3. DETAILS MUST BE FOLLOWED TO ENSURE PROPER TESTING RESULTS.
4. ALL WOOD TO STEEL CONNECTIONS MUST BE SEPARATED BY SILL PLATE GASKET. CHEMICALLY TREATED LUMBER WILL CAUSE STEEL CORROSION.
5. ($3\frac{5}{8}$ ") FIBERGLASS INSULATION MUST BE INSTALLED TIGHT BETWEEN JOIST IN THE CEILING CAVITY.
6. SCP SHEATHING TO BE USED ON SCP BUILDING ONLY. USE OSB ELSWHERE. SHINGLES MUST BE STAPLED IN LIEU OF BEING NAILED TO PREVENT SHEATHING BLOW-OUT.
7. MINERAL WOOL INSULATION MUST BE INSTALLED DURING FRAMING IN ALL CLOSED-OFF STUD CAVITIES.
8. RESILIENT CHANNELS ALONG THE WALLS MUST BE INSTALLED WITH THE FLEXIBLE END TURNED UP AND THE SCREWS ON THE BOTTOM.
9. THE SECOND LAYER OF CEILING DRYWALL IS TO BE ADDED AFTER THE FIRST ROUND OF TESTING.

NAME AND COMPANY	SIGNATURE FOR APPROVAL	APPROVAL DATE
GREG RALPH DIETRICH INDUSTRIES INCORPORATED		
MIKE WHITTICAR DIETRICH BUILDING SYSTEMS		
RANDY DAUDET DDG		
TOM SHEPPARD USG		
BERT PASSALACQUA DIETRICH INDUSTRIES INCORPORATED		
GEORGE HARAKAL DIETRICH INDUSTRIES INCORPORATED		
JIM REICHERTS USG		
TIM TONYAN USG		

Project Name: SOUND CHAMBER
Location: HAMMOND, IN
Project Number: 04-1229
Drawn by: CAD2
Approved by:
Date: 11.11.04
Revisions:
1-20-05 **FOR CONSTRUCTION**
11.30.05 **AS-BUILT**

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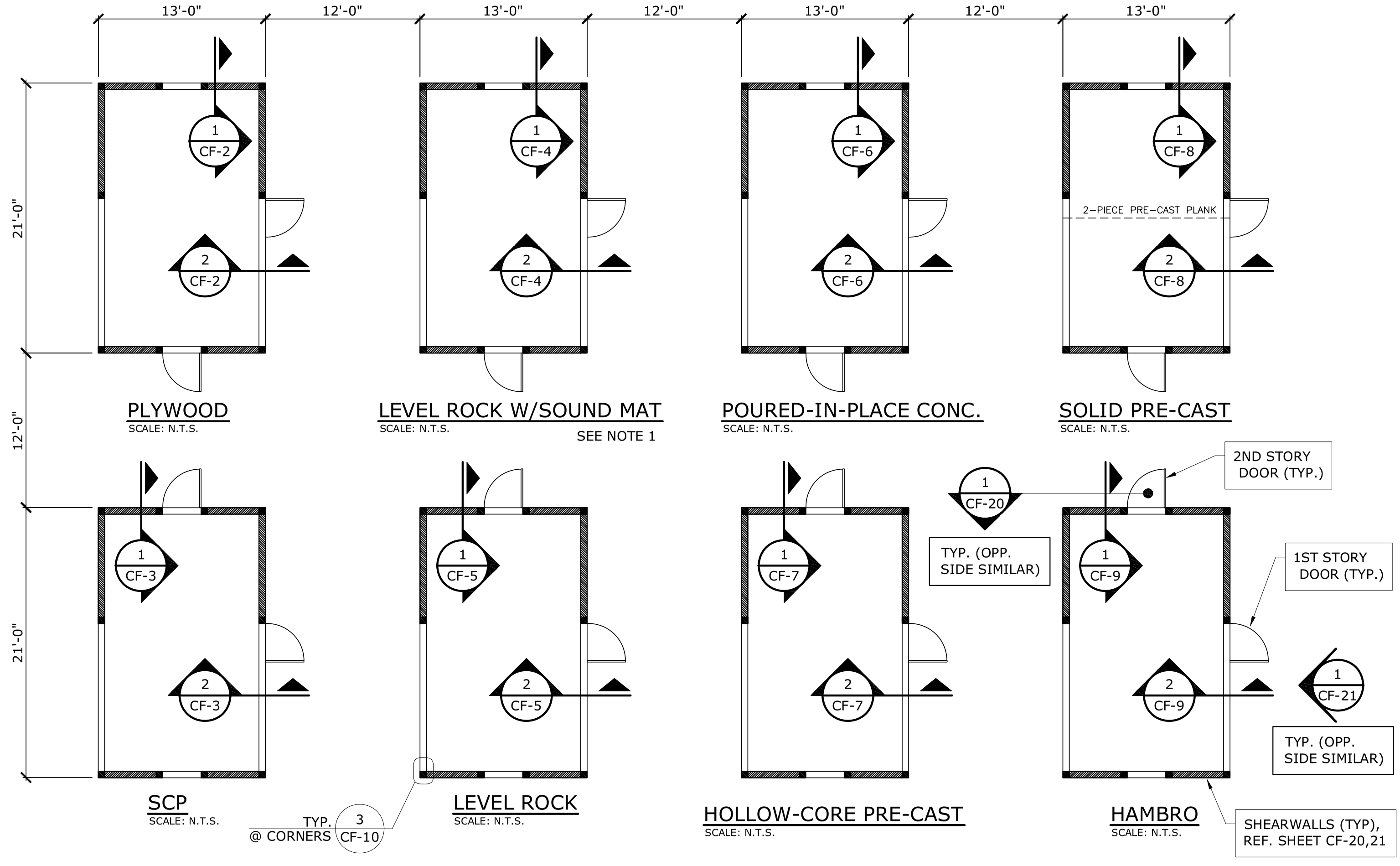
Sheet Description: INDEX AND APPROVAL
Sheet: CF 1 of **



NOTE:
1. DO NOT PROCEED WITH THE INSTALLATION OF LEVELROCK AND SOUND MAT UNTIL RELEASE BY USG..

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Location	HAMMOND, IN
Project Number	04-1229
Drawn by	CAD1
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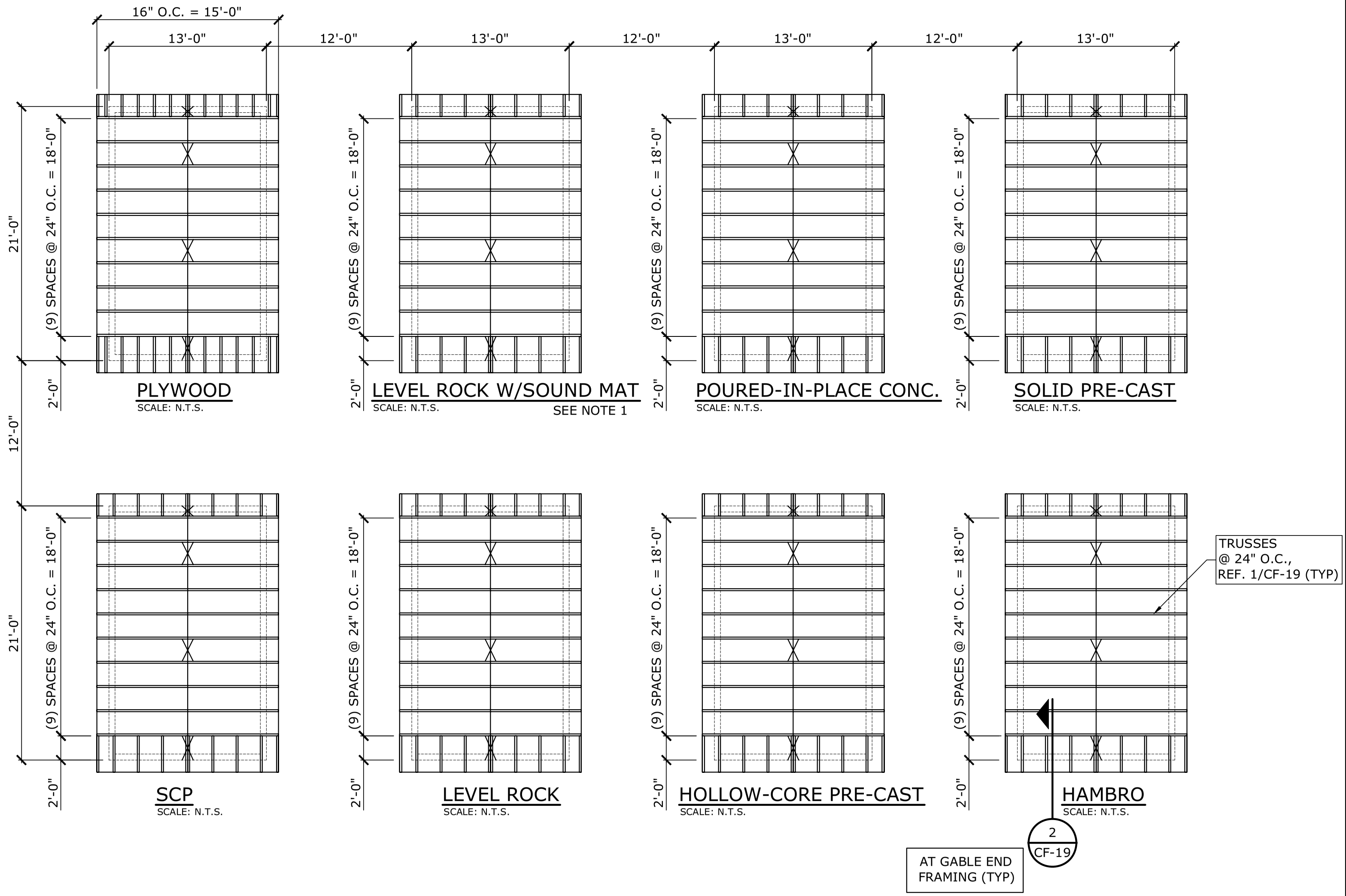
Sheet Description: PLATFORM LAYOUT

Sheet: CF 1a of **



NOTE:
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Location: HAMMOND, IN
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Drawn by: CAD1
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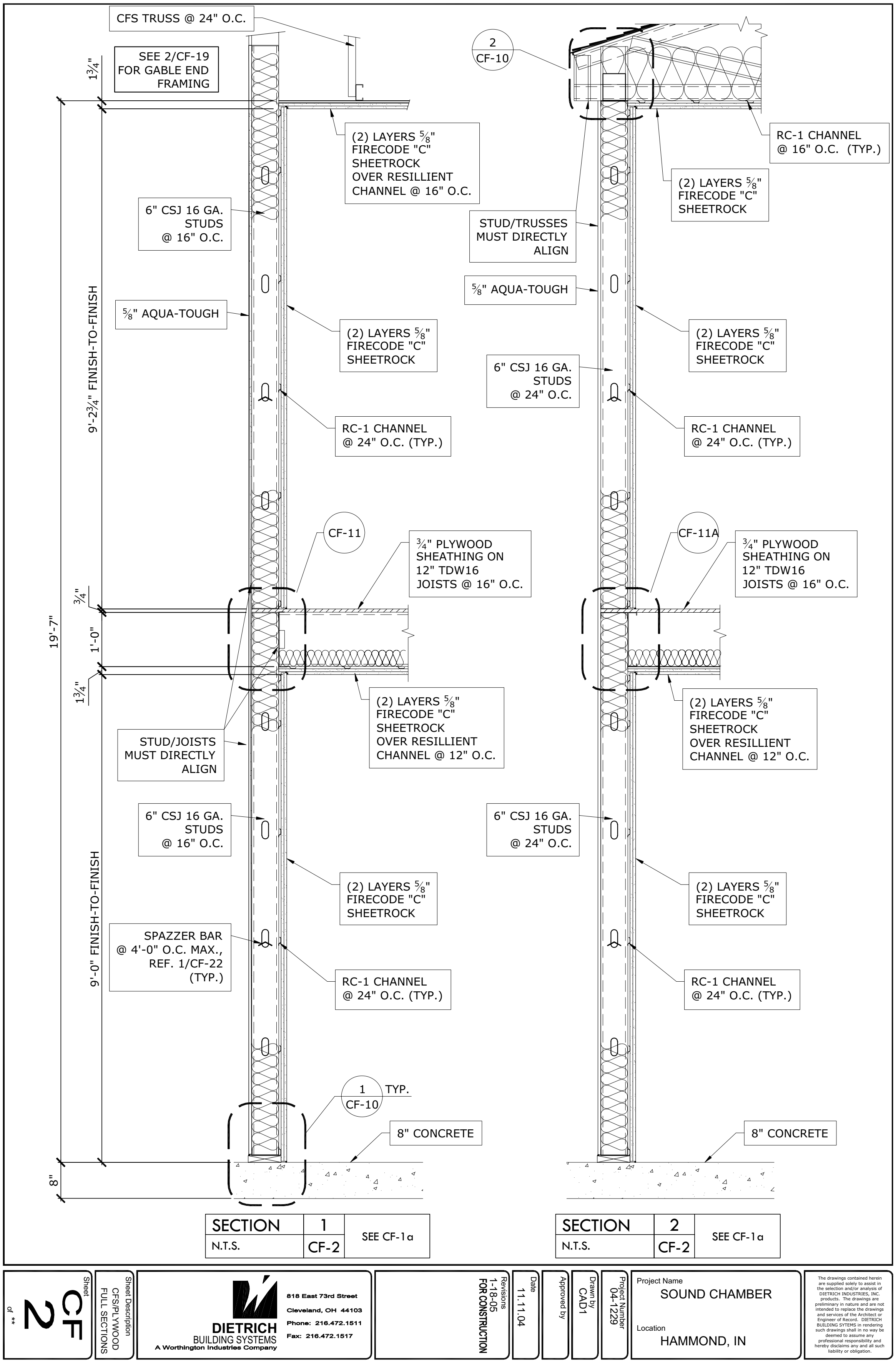
Sheet Description: ROOF FRAMING
Sheet: CF 1C of **



CFS TRUSS @ 24" O.C.
SEE 2/CF-19 FOR GABLE END FRAMING
1¾"
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK OVER RESILLIENT CHANNEL @ 16" O.C.
6" CSJ 16 GA. STUDS @ 16" O.C.
⅝" AQUA-TOUGH
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK
9'-2¾" FINISH-TO-FINISH
RC-1 CHANNEL @ 24" O.C. (TYP.)
CF-11
¾" PLYWOOD SHEATHING ON 12" TDW16 JOISTS @ 16" O.C.
19'-7"
¾"
1'-0"
1¾"
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK OVER RESILLIENT CHANNEL @ 12" O.C.
STUD/JOISTS MUST DIRECTLY ALIGN
6" CSJ 16 GA. STUDS @ 16" O.C.
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK
9'-0" FINISH-TO-FINISH
SPAZZER BAR @ 4'-0" O.C. MAX., REF. 1/CF-22 (TYP.)
RC-1 CHANNEL @ 24" O.C. (TYP.)
1 CF-10 TYP.
8" CONCRETE
8"
SECTION 1 N.T.S. CF-2 SEE CF-1a
2 CF-10
RC-1 CHANNEL @ 16" O.C. (TYP.)
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK
STUD/TRUSSES MUST DIRECTLY ALIGN
⅝" AQUA-TOUGH
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK
6" CSJ 16 GA. STUDS @ 24" O.C.
RC-1 CHANNEL @ 24" O.C. (TYP.)
CF-11A
¾" PLYWOOD SHEATHING ON 12" TDW16 JOISTS @ 16" O.C.
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK OVER RESILLIENT CHANNEL @ 12" O.C.
6" CSJ 16 GA. STUDS @ 24" O.C.
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK
RC-1 CHANNEL @ 24" O.C. (TYP.)
8" CONCRETE
SECTION 2 N.T.S. CF-2 SEE CF-1a
Sheet CF 2 of **
Sheet Description CFS/PLYWOOD FULL SECTIONS
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Drawn by CAD1
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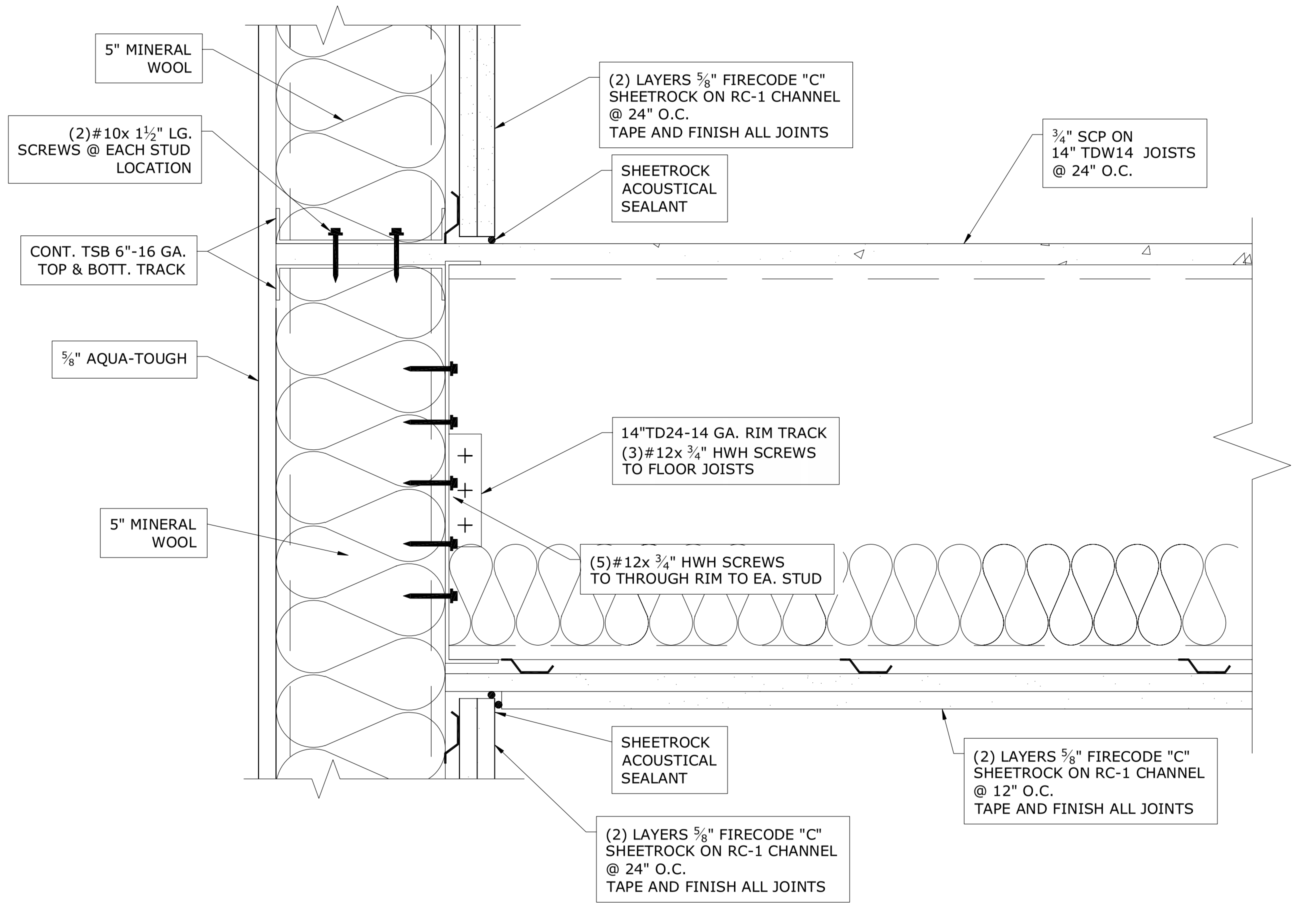

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SOUND CHAMBER

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Sheet
CF
12
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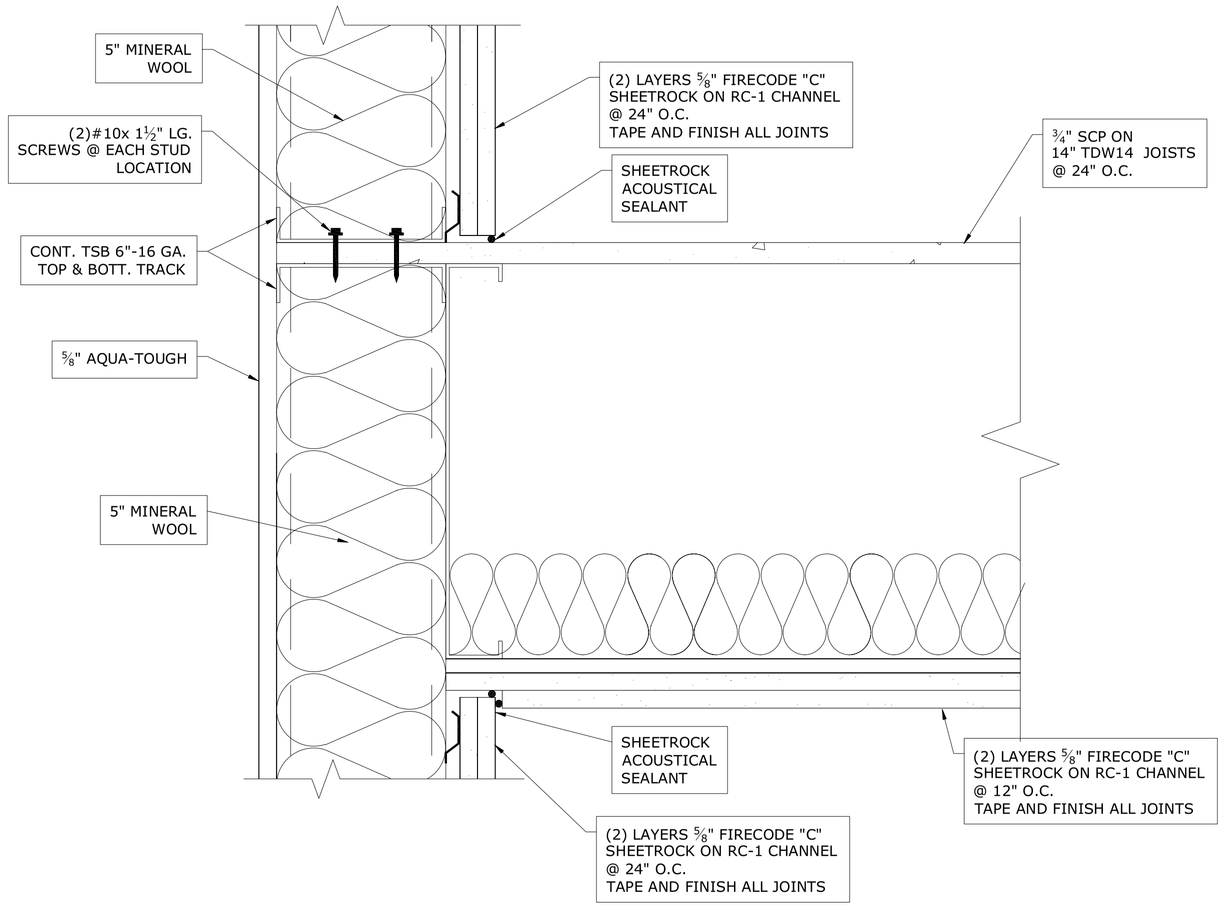

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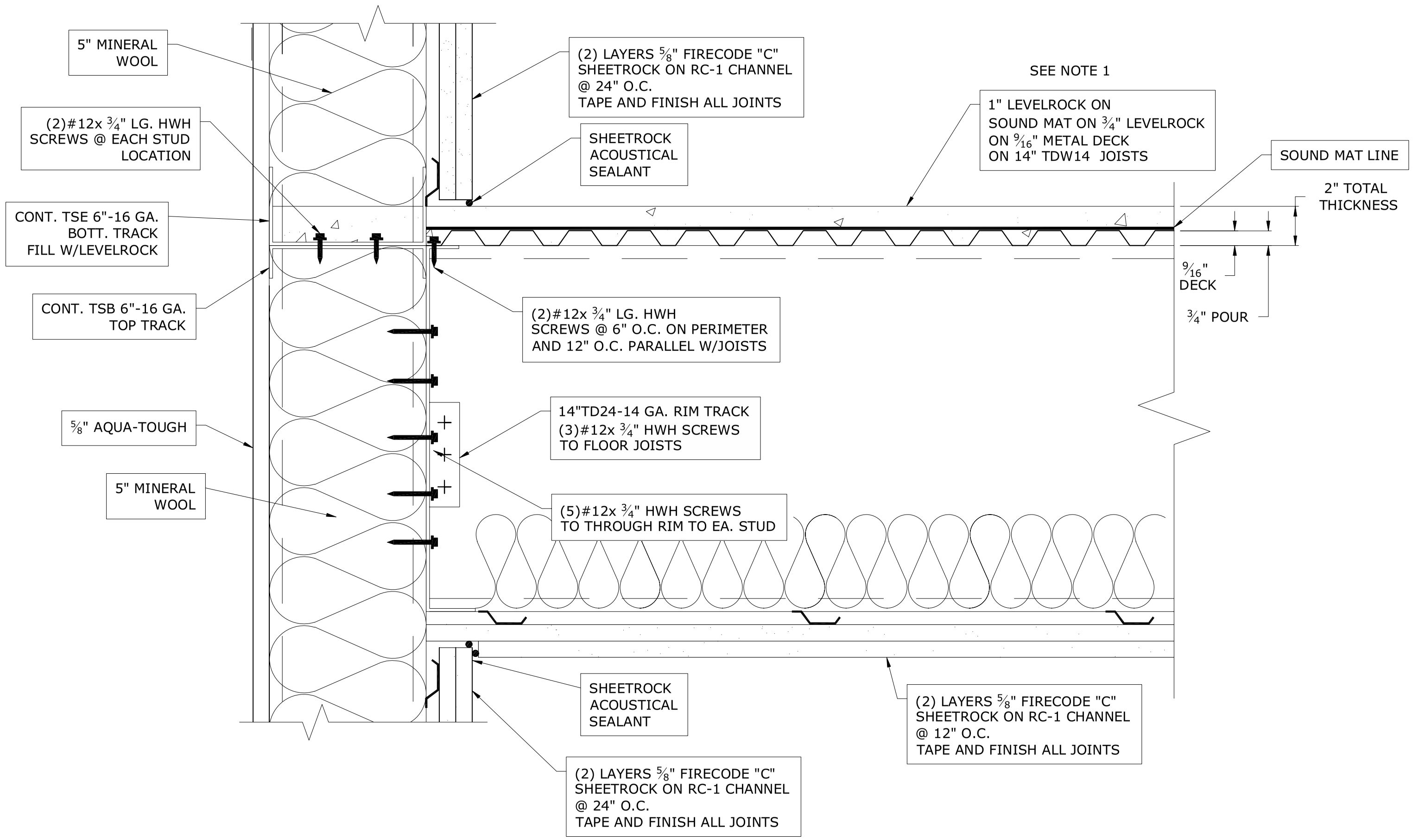
Sheet: CF 12A of **



NOTE:
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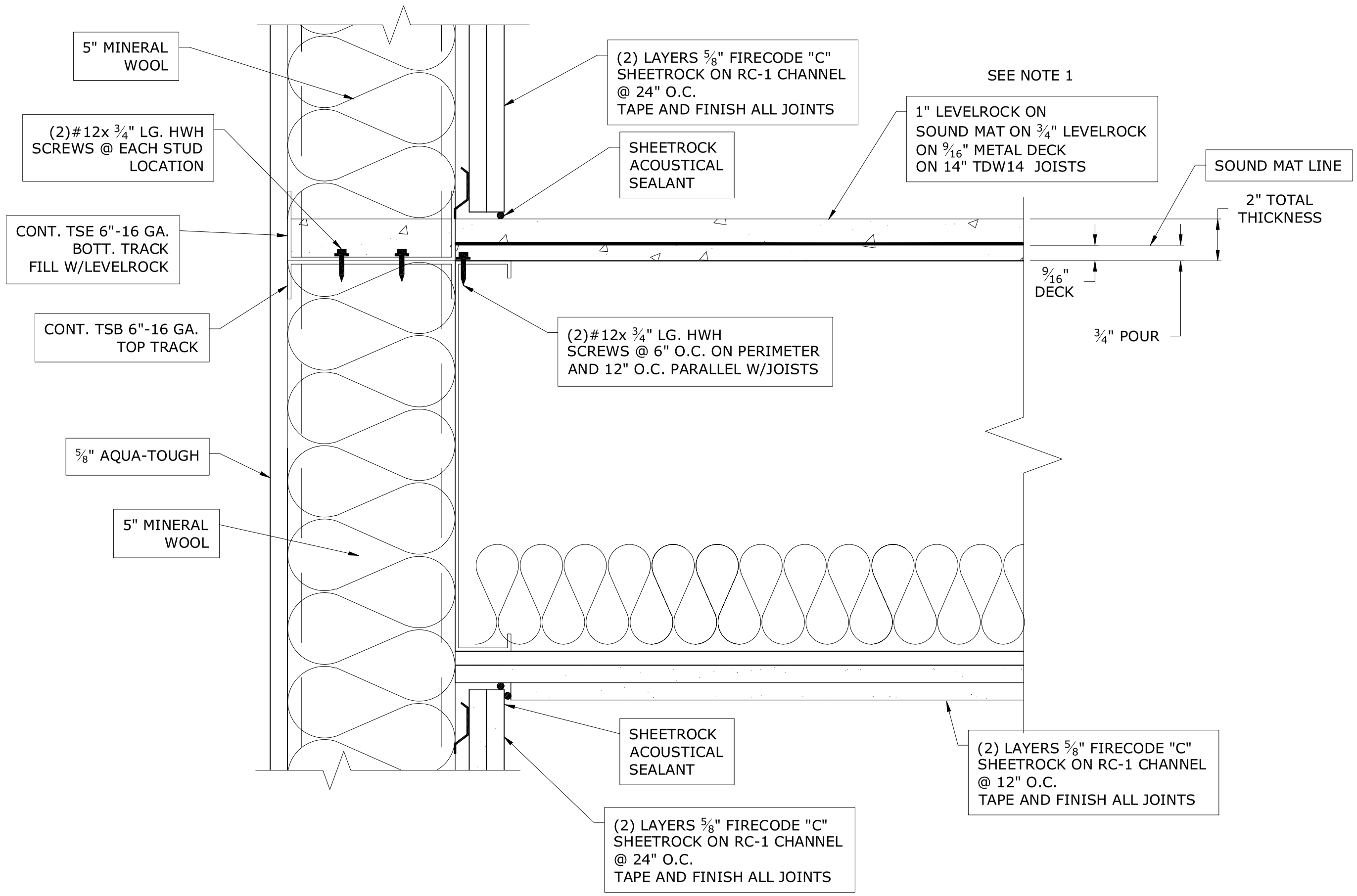
Sheet Description: CFS/LEVELROCK W/SOUND MAT
Sheet: CF 13 of **



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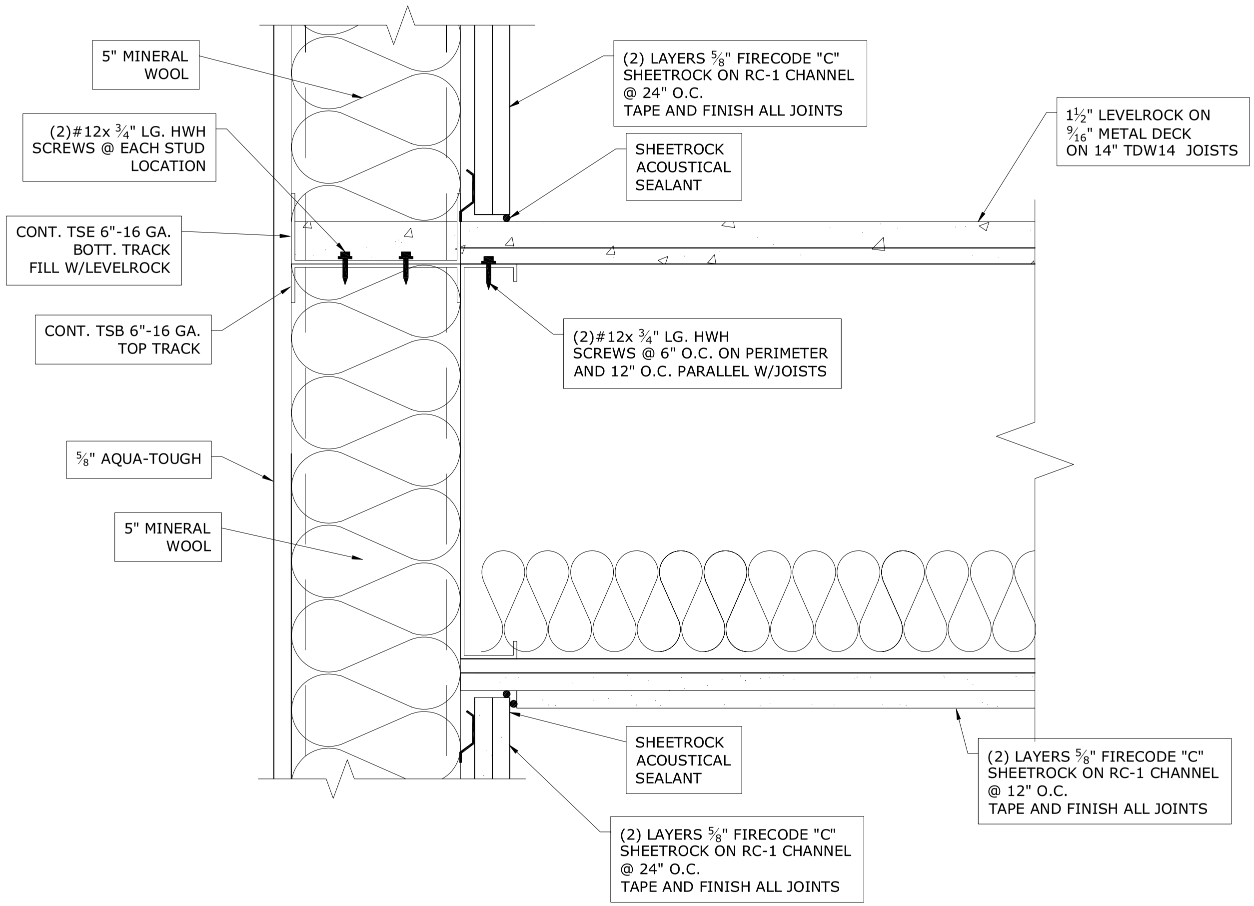
Sheet Description
CFS/LEVELROCK W/SOUND MAT

Sheet
CF 13A
of **



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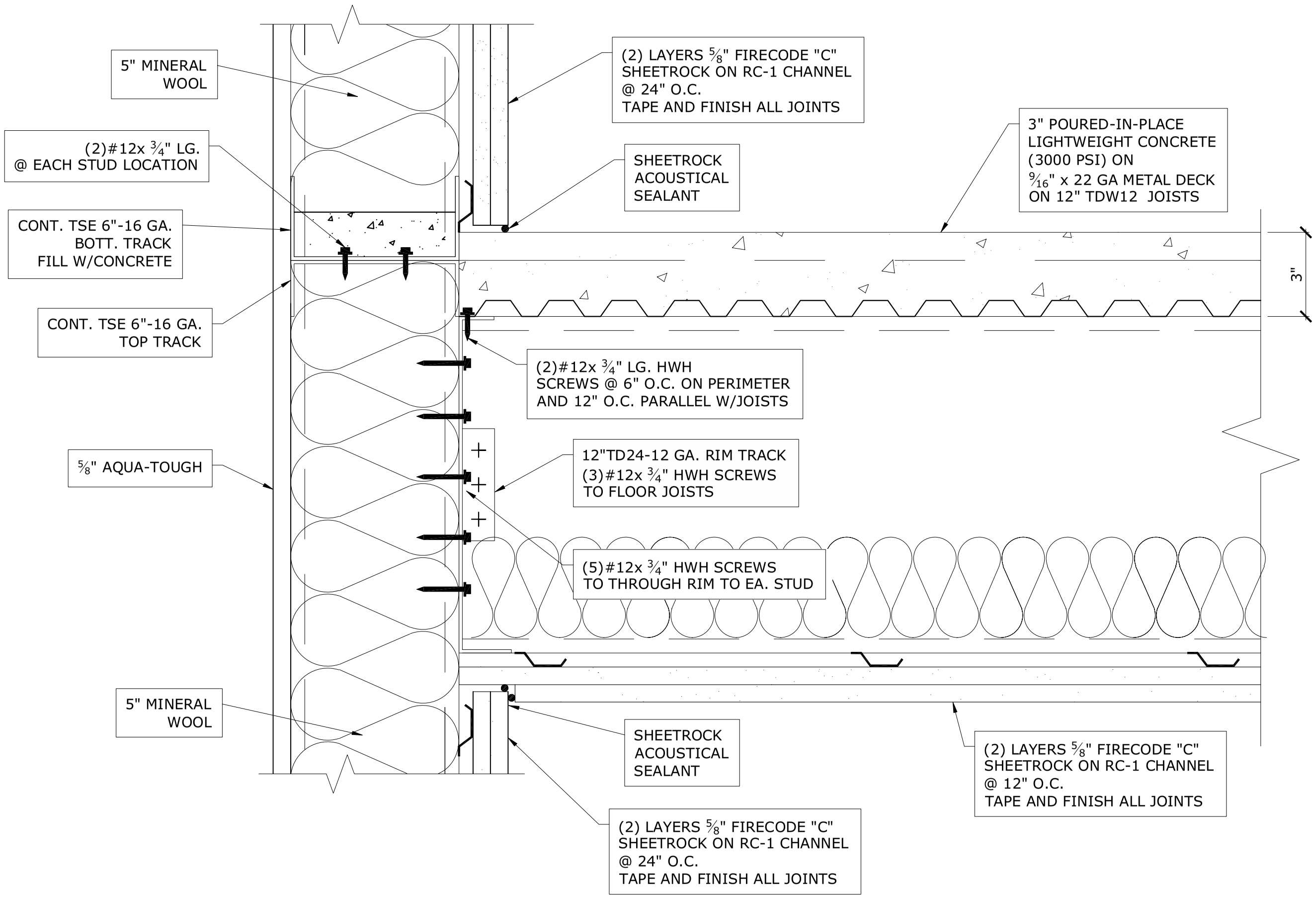
Sheet Description: CFS/LEVELROCK AT FLOOR
Sheet: CF 14A of **



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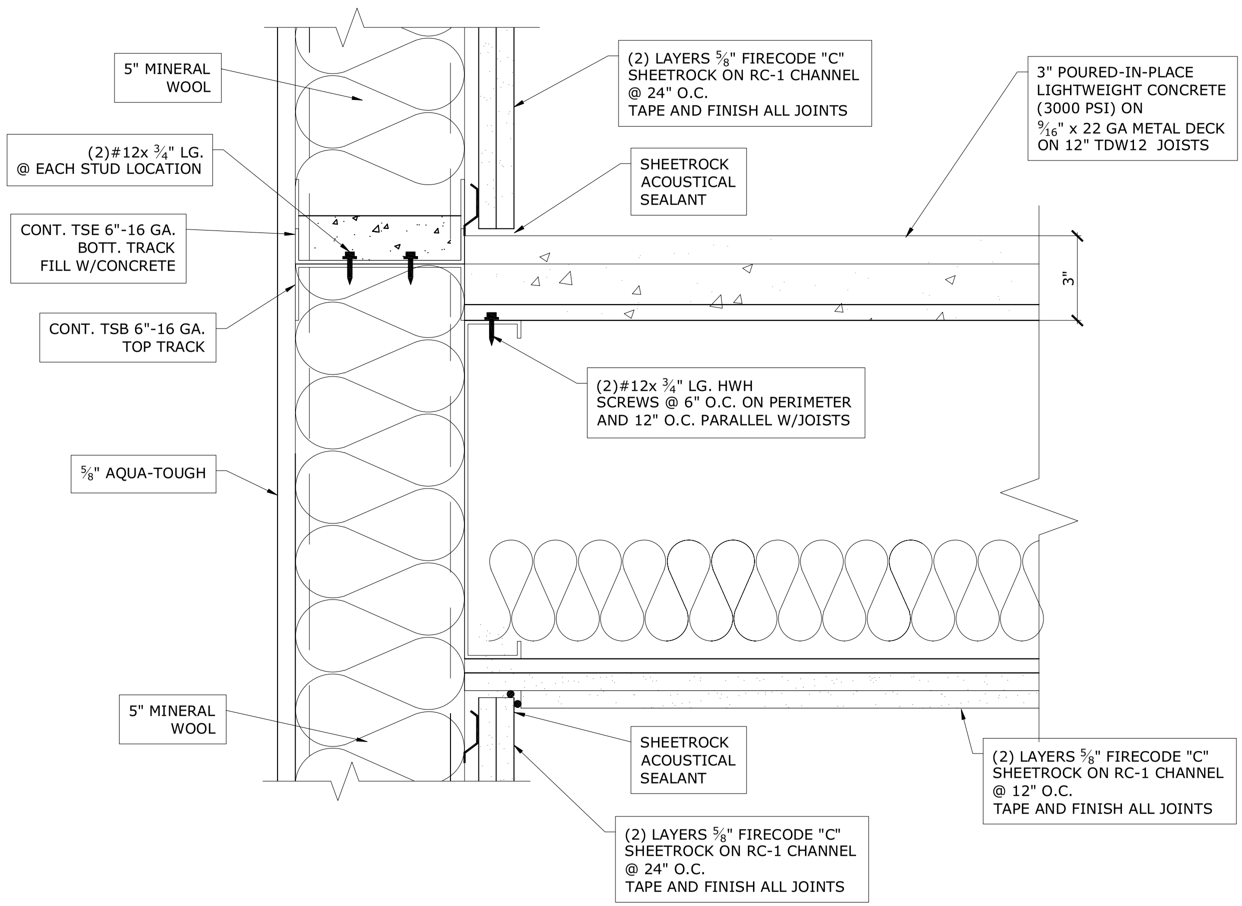
Sheet Description: CFS/POURED-IN-PLACE CONC.

Sheet: CF 15 of **



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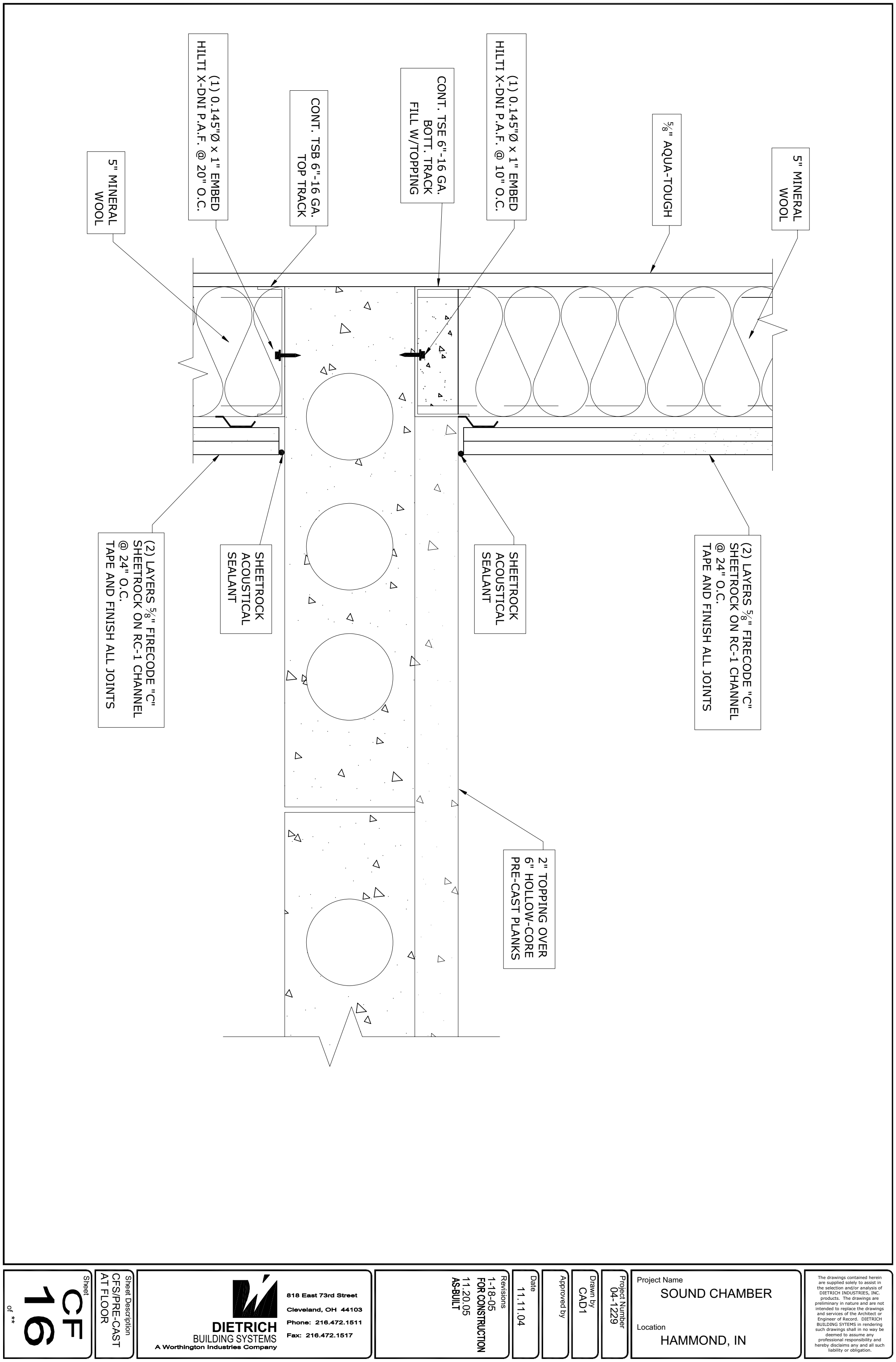
Sheet Description: CFS/POURED-IN-PLACE CONC.

Sheet: CF 15A of **


5" MINERAL WOOL
(1) 0.145"Ø x 1" EMBED HILTI X-DNI P.A.F. @ 20" O.C.
CONT. TSB 6"-16 GA. TOP TRACK
CONT. TSE 6"-16 GA. BOTT. TRACK FILL W/TOPPING
(1) 0.145"Ø x 1" EMBED HILTI X-DNI P.A.F. @ 10" O.C.
5/8" AQUA-TOUGH
5" MINERAL WOOL
(2) LAYERS 5/8" FIRECODE "C" SHEETROCK ON RC-1 CHANNEL @ 24" O.C. TAPE AND FINISH ALL JOINTS
SHEETROCK ACOUSTICAL SEALANT
SHEETROCK ACOUSTICAL SEALANT
(2) LAYERS 5/8" FIRECODE "C" SHEETROCK ON RC-1 CHANNEL @ 24" O.C. TAPE AND FINISH ALL JOINTS
2" TOPPING OVER 6" HOLLOW-CORE PRE-CAST PLANKS
Sheet
CF 16
of **
Sheet Description
CFS/PRE-CAST AT FLOOR
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Drawn by
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Project Name
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Location
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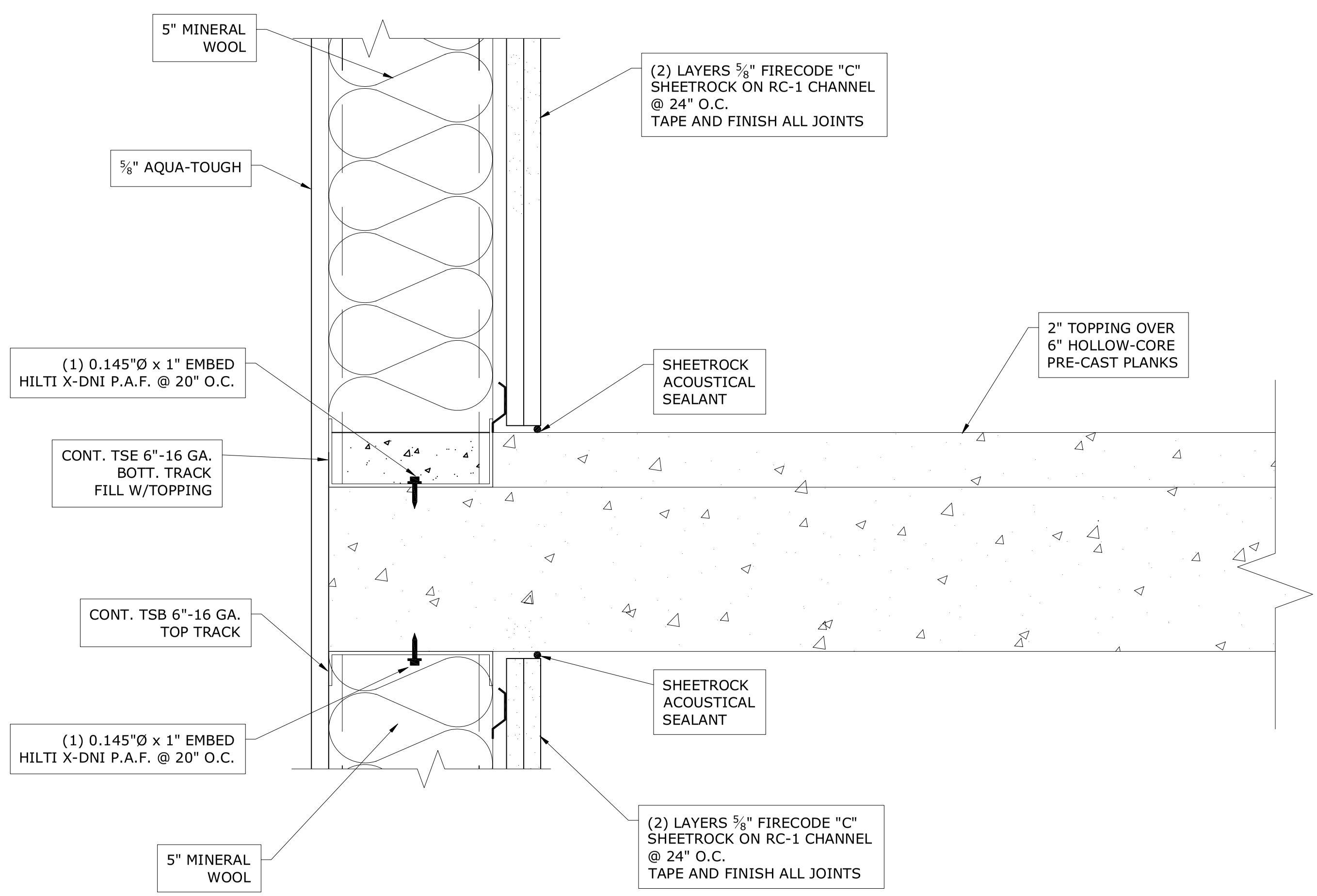

Project Name: SOUND CHAMBER
Location: HAMMOND, IN
Project Number: 04-1229
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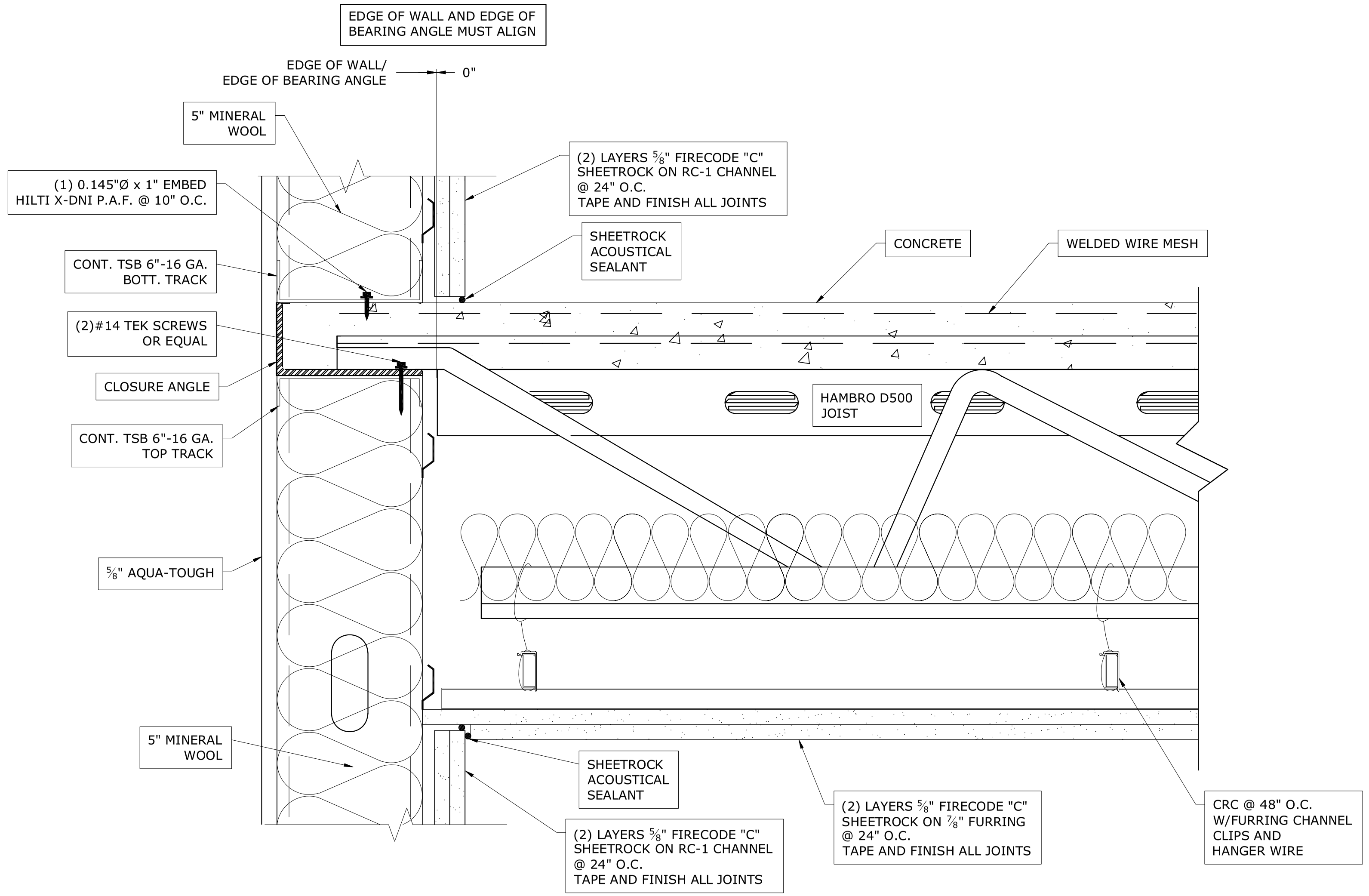
Sheet Description: CFS/PRE-CAST AT FLOOR
Sheet: CF 16A of **



Project Name
SOUND CHAMBER
Location
HAMMOND, IN

Project Number
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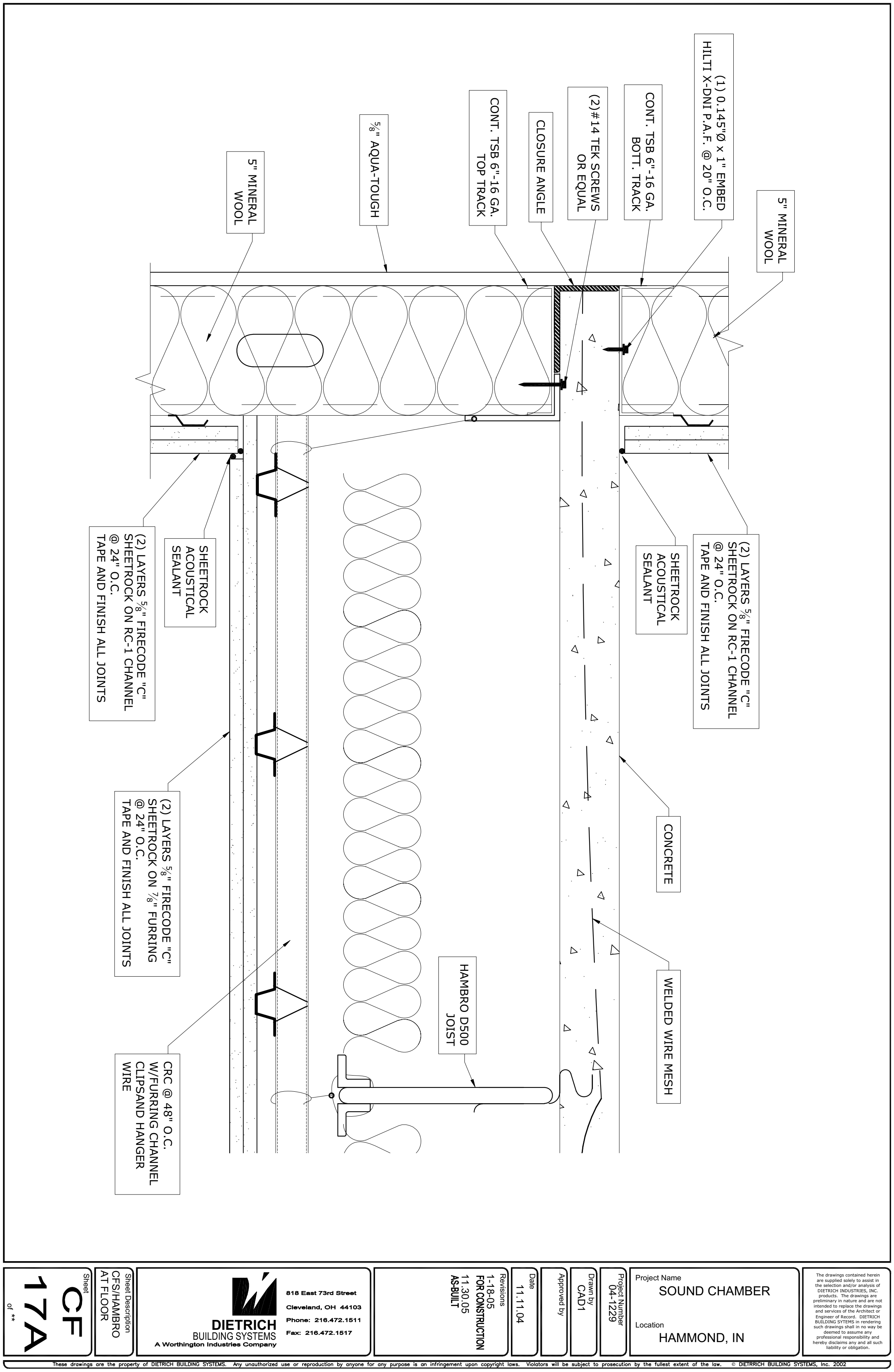
Sheet Description
CFS/HAMBRO
AT FLOOR

Sheet
CF
17
of **



5" MINERAL WOOL
(1) 0.145"Ø x 1" EMBED HILTI X-DNI P.A.F. @ 20" O.C.
CONT. TSB 6"-16 GA. BOTT. TRACK
(2)#14 TEK SCREWS OR EQUAL
CLOSURE ANGLE
CONT. TSB 6"-16 GA. TOP TRACK
⅝" AQUA-TOUGH
5" MINERAL WOOL
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK ON RC-1 CHANNEL @ 24" O.C. TAPE AND FINISH ALL JOINTS
SHEETROCK ACOUSTICAL SEALANT
SHEETROCK ACOUSTICAL SEALANT
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK ON RC-1 CHANNEL @ 24" O.C. TAPE AND FINISH ALL JOINTS
CONCRETE
(2) LAYERS ⅝" FIRECODE "C" SHEETROCK ON ⅞" FURRING @ 24" O.C. TAPE AND FINISH ALL JOINTS
WELDED WIRE MESH
HAMBRO D500 JOIST
CRC @ 48" O.C. W/FURRING CHANNEL CLIPSAND HANGER WIRE
Sheet
CF 17A
of **
Sheet Description
CFS/HAMBRO AT FLOOR
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A Worthington Industries Company
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Project Name
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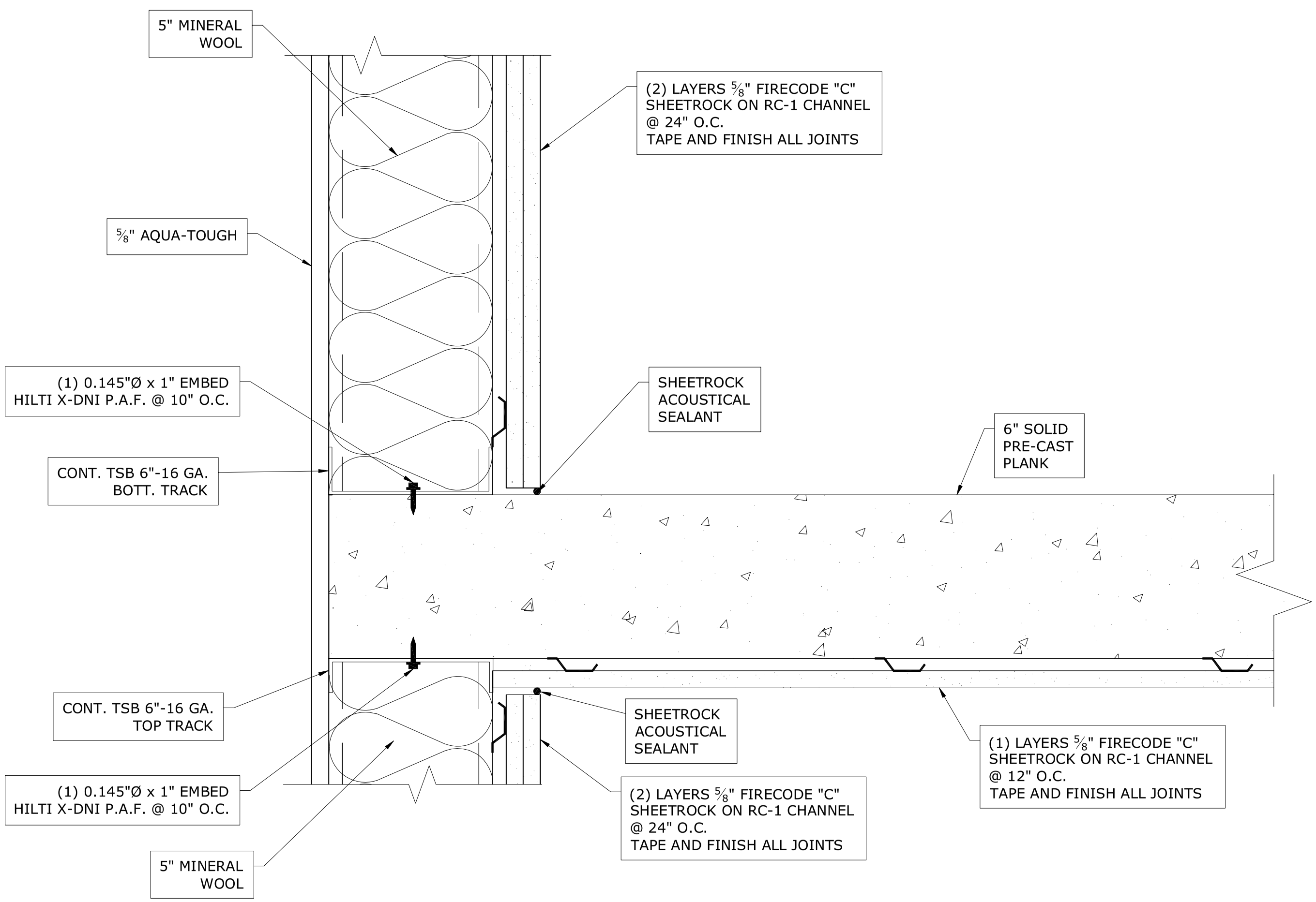

Project Name: SOUND CHAMBER

Location: HAMMOND, IN

Project Number: 04-1229

Drawn by: CAD1

Approved by:

Date: 11.11.04

Revisions:
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11.30.05 **AS-BUILT**

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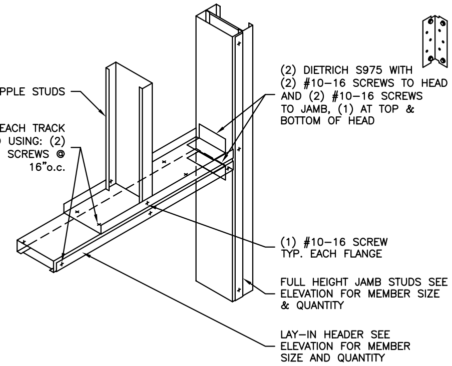
Sheet Description: CFS/EPICORE AT FLOOR

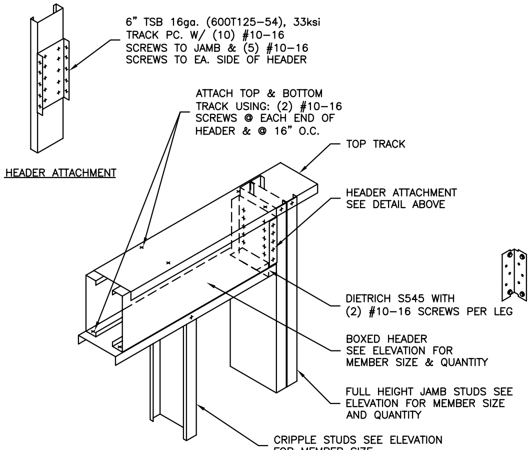
Sheet: CF 18 of **

DETAIL NOT USED

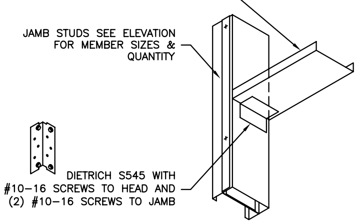
1 NOT USED



2 LAY-IN HEADER CONNECTION



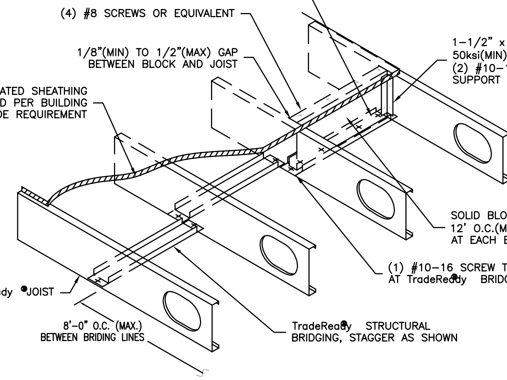
3 BOXED LINTEL CONNECTION



4 HEAD TRACK CONNECTION

TABLE T.1

JOIST DEPTH	CLIP LENGTH
7-1/4" - 8"	5"
9-1/4" - 10"	7"
11-1/4" - 14"	9"



5 TradeReady® STRUCTURAL BRACING DETAIL

Project Name: SOUND CHAMBER
Location: HAMMOND, IN
Project Number: 04-1229
Drawn by: JGH
Approved by:
Date: 10.1.04
Revisions: 1.19.05 FOR CONSTRUCTION

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Sheet Description: DETAILS
Sheet: CF 23 of **

Appendix B – Laboratory and In Situ Testing Data

Table B-1: Complete Processed Data Set from Laboratory Testing in Balloon Framing Condition

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	β_2 (%)	Δ_{center} (mm)	a_{RMS} (g)
	Heel Drop				Sandbag						
LF14.5A	25.8	33.1	7.3%	1.44	25.3	32.7	4.3%	0.500	2.3%*	0.52	0.015
LF14.5B	22.4	25.0	2.3%	1.19	22.5	25.1	3.2%	0.206	3.0%*	0.44	0.016
LF14.5C	25.9	34.0	7.5%	0.69	26.3	33.2	2.1%	1.202	2.0%*	0.59	-
LF14.5D	19.7	24.1	4.7%	1.34	19.7	24.2	2.1%	1.743	2.2%	0.34	0.014
LF14.5D$_i$	24.0	29.2	3.8%	2.05	24.1	28.8	1.7%	0.264	2.1%*	0.38	0.022
LF14.5E	17.7	22.6	3.1%	0.70	17.7	22.5	1.0%	0.902	2.3%	0.22	0.029
LF14.5F	16.0	22.6	3.8%	0.72	16.1	22.5	0.8%	0.457	3.0%	0.18	0.037
LF17.0A	14.7	20.2	4.4%	0.89	14.9	19.1	0.8%	0.574	4.0%	0.3	0.011
LF17.0B	14.7	19.6	3.9%	0.78	14.9	19.7	0.8%	0.502	3.6%	0.27	0.012
LF17.0C	14.3	18.4	3.6%	0.60	14.3	18.3	0.8%	0.441	3.0%	0.25	0.014
LF17.0D	14.2	19.9	3.4%	0.68	14.3	19.9	0.7%	0.404	3.4%	0.22	0.013
LF19.5A	11.9	17.0	3.1%	0.76	12.0	16.9	0.7%	0.552	2.9%	0.33	0.014
LF19.5A$_i$	12.7	18.6	4.4%	0.84	12.7	17.4	1.4%	0.666	2.8%	0.37	-
LF19.5A$_{ii}$	13.2	24.3	4.5%	1.01	13.2	24.0	0.6%	0.791	4.1%	0.35	-
LF19.5A$_{iii}$	12.9	22.7	4.5%	0.86	13.0	23.0	0.7%	0.539	3.4%	0.3	-
LF19.5A$_{iv}$	10.3	14.8	2.6%	0.56	10.4	14.8	0.6%	0.411	3.1%	0.33	-
LF19.5B	11.4	16.8	3.9%	0.76	11.4	16.8	1.6%	0.491	3.4%	0.28	0.011
LF19.5B$_i$	11.9	17.6	3.0%	0.92	12.0	17.5	0.4%	0.798	2.9%	-	-
LF19.5B$_{ii}$	12.6	18.9	4.3%	0.93	12.7	25.0	1.6%	0.576	3.6%	0.29	-
LF19.5B$_{iii}$	12.3	23.8	4.0%	0.75	12.5	23.4	0.6%	0.45	3.7%	0.26	-
LF19.5B$_{iv}$	9.9	14.8	2.4%	0.62	10.0	14.9	0.6%	0.412	3.2%	-	-
LF21.8A	11.7	16.9	3.4%	0.61	11.8	16.0	1.2%	0.532	3.6%	0.28	0.011

* indicates β2 from Sandbag, otherwise Heel Drop data is reported

Table B-2: Complete Processed Data Set from Laboratory Testing in Platform Framing Condition

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	β_2 (%)	Δ_{center} (mm)	a_{RMS} (g)
	Heel Drop				Sandbag						
LF14.5A	-	-	-	-	17.9	29.8	3.7%	0.36	3.7%*	0.67	0.013
LF14.5B	17.2	18.7	4.6%	1.26	17.2	18.8	3.8%	0.21	1.9%*	0.48	0.014
LF14.5C	16.4	27.0	7.6%	0.97	16.4	27.8	3.7%	1.061	2.2%*	0.62	-
LF14.5D	16.7	22.7	7.0%	1.52	16.9	22.0	2.6%	1.581	2.6%	0.37	0.012
LF14.5E	15.8	21.7	5.3%	0.65	16.2	22.2	1.7%	0.726	3.9%	0.24	0.031
LF14.5F	14.7	20.8	3.4%	0.60	14.8	22.0	1.1%	0.438	2.9%	0.17	0.039
LF17.0A	13.4	19.3	4.0%	0.62	13.6	19.4	1.5%	0.602	3.4%	0.32	0.010
LF17.0B	13.1	19.3	5.7%	0.69	13.3	19.3	2.4%	0.605	4.7%	0.29	0.011
LF17.0C	12.7	19.8	4.0%	0.58	13.4	18.8	1.0%	0.441	3.5%	0.25	0.010
LF17.0D	13.1	20.2	4.1%	0.58	13.4	20.2	1.4%	0.387	3.2%	0.23	0.010
LF19.5A	11.7	17.2	3.8%	0.58	11.8	17.3	1.1%	0.549	3.2%	0.34	0.013
LF19.5A$_{iv}$	10.3	15.2	2.5%	0.47	10.6	15.3	0.8%	0.413	2.2%	0.34	-
LF19.5B	10.8	16.9	3.5%	0.74	11.1	17.0	0.8%	0.578	3.4%	0.28	0.014
LF21.8A	9.9	15.4	3.6%	0.57	10.1	15.5	2.3%	0.546	3.5%	0.31	0.008

* indicates β2 from Sandbag, otherwise Heel Drop data is reported

Table B-3: Complete Processed Data Set from Laboratory Testing in Simple Support Condition

Floor Name	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	f_1 (Hz)	f_2 (Hz)	β_1 (%)	a_{peak} (g)	β_2 (%)	Δ_{center} (mm)	a_{RMS} (g)
	Heel Drop				Sandbag						
LF14.5A	19.5	30.0	6.7%	1.48	19.1	27.4	5.5%	0.4	3.0%*	0.55	0.015
LF14.5B	17.6	22.3	3.7%	1.08	17.2	21.4	2.9%	0.222	2.8%*	0.54	0.018
LF14.5B_i	16.4	24.8	2.4%	0.96	-	-	-	-	-	-	-
LF14.5C	17.7	25.8	7.8%	0.93	17.7	26.0	2.3%	1.073	1.7%*	0.71	-
LF14.5D	16.3	22.9	7.7%	1.20	16.2	22.4	2.8%	1.451	2.4%	0.4	0.014
LF14.5E	15.2	21.2	5.7%	0.56	15.7	21.1	2.5%	0.708	4.0%	0.25	0.029
LF14.5F	14.3	21.0	3.2%	0.55	14.6	21.2	1.3%	0.444	3.1%	0.2	0.047
LF17.0A	13.2	18.8	4.8%	0.72	13.5	17.9	1.9%	0.609	4.1%	0.34	0.011
LF17.0B	13.1	18.0	4.4%	0.83	13.3	18.1	1.4%	0.541	3.4%	0.32	0.010
LF17.0C	12.7	18.3	3.2%	0.63	12.8	18.4	0.9%	0.406	3.6%	0.26	0.014
LF17.0D	12.9	18.3	4.5%	0.68	13.2	18.6	1.1%	0.407	3.3%	0.24	0.009
LF19.5A	11.2	16.3	4.9%	0.64	11.4	16.4	1.9%	0.548	3.7%	0.35	0.010
LF19.5A_{iv}	9.9	14.6	3.5%	0.52	10.1	14.7	1.1%	0.449	2.8%	0.35	-
LF19.5B	10.4	16.1	3.7%	0.76	10.6	16.3	1.2%	0.65	3.8%	0.3	0.012
LF21.8A	9.7	14.3	3.6%	0.53	9.9	14.5	1.9%	0.506	3.6%	0.34	0.009

* indicates β2 from Sandbag, otherwise Heel Drop data is reported

Table B-4: Complete Processed Data Set from In Situ Testing in Balloon Framing Condition

Floor Name	**f_1 (Hz)**	**f_2 (Hz)**	**β_1 (%)**	**a_{peak} (g)**	**f_1 (Hz)**	**f_2 (Hz)**	**β_1 (%)**	**a_{peak} (g)**	**β_2 (%)**	**Δ_{center} (mm)**	**a_{RMS} (g)**
	Heel Drop				Sandbag						
CG601	13.6	23.4	-	0.74	14.4	23.1	5.4%	-	-	0.46	0.007
CG604	15.4	24.0	15.2%	0.70	16.3	23.1	8.8%	-	-	0.28	0.006
CG805	14.4	22.4	8.0%	0.57	15.2	22.8	5.1%	-	8.2%	-	-
CGMH6	15.0	22.5	11.3%	0.71	15.7	23.8	10.7%	-	8.8%	0.36	-
CGMH7	15.4	24.5	12.3%	1.15	16.6	25.6	8.5%	-	7.1%	-	-
CW707	16.1	20.4	7.3%	0.45	16.1	20.2	6.5%	-	9.8%	-	0.003
CW708	18.7	23.2	7.3%	0.37	18.7	22.9	5.6%	-	7.7%	-	0.005
CW709	9.9	13.1	5.8%	0.35	9.9	12.9	4.1%	-	-	-	0.005
CW805	11.8	24.3	8.1%	0.37	11.9	23.0	12.6%	-	-	-	-
DDG1	11.9	21.1	11.8%	1.10	12.0	21.6	4.7%	-	8.9%	0.39	0.014
DDG2	13.2	-	10.8%	0.54	14.0	-	6.7%	-	-	0.37	0.006
DDG3	13.1	26.8	8.4%	0.66	13.2	27.1	3.5%	-	-	0.2	0.008
DDG4	16.1	-	6.1%	0.45	16.1	-	2.9%	-	-	0.08	0.005
OK401	17.8	23.5	4.5%	0.67	22.3	28.6	8.4%	-	-	-	0.004
OK402	16.6	27.1	6.0%	0.43	23.7	27.9	5.8%	-	-	-	0.003
OK403	33.2	-	-	0.72	32.8	45.7	-	-	-	-	0.002

Appendix C – Material and Section Properties Used for Calculations

Table C-1: Material and Section Properties Used for Calculations

Joist	**Mass (kg/m)**	**t (mm)**	**d (mm)**	**Area (mm^2)**	**E (GPa)**	**G (GPa)**	**I (mm^4)**	**R (mm)**	**Cw (mm^6)**	**J (mm^4)**
TDW14	5.80	1.81	305	769	203	77	$9.133*10^6$	113	$3.768*10^9$	840
TDW16	4.64	1.44	305	615	203	77	$7.358*10^6$	114	$3.079*10^9$	424
CSJ16	2.06	1.44	92.1	272	203	77	$3.634*10^5$	15	$1.206*10^8$	188
Sheathing	**Density (kg/m^3)**	**t (mm)**	**E (GPa)**	**G (GPa)**	**EI_{joist} (Nm)**	**EI_{trans} (Nm)**				
UFS	9774	0.749	203	77	0	2113				
FC	1201	19.1	7.58	1.52[a]	4335	4335				
OSB	650	19.1	3.14	0.200	720	2800				
Topping	**Density (kg/m^3)**	**t (mm)**	**E (GPa)**	**G (GPa)**	**EI_{joist} (Nm)**	**EI_{trans} (Nm)**				
LR	1922	19.1	22.1[c]	0	12635	12635				
LR	1922	31.8[b]	22.1[c]	0	19369	58960				
Ceiling	**Density (kg/m^3)**	**t (mm)**								
Type X		19.1								
Type C		19.1								

a – assumed to be 0.2*E from SCH recommendation

b – taken as average depth, moment of inertia assumed rectangular section

c – 1.35*E, recommended for dynamic behavior of concrete by ATC

Appendix D – Design Example Using Proposed Procedure

<table>
<tr><td colspan="3">This is a discussion of the worked example showing the methods from the ATC and AISC Design Guides, with modifications based on the recommendations from this thesis, and comments based on assumption, applicability and calculations. A floor with cold-formed steel deck is used in order to illustrate the procedure for incorporating the flutes and varying cross-section.</td></tr>
<tr><td></td><td>ATC Design Guide</td><td>AISC Design Guide</td></tr>
<tr><td colspan="3">Design Procedure Step 1: Determine Fundamental Frequency</td></tr>
<tr><td>Expression for Fundamental Frequency considering joist panel only:</td><td>$$f_n = 0.18\sqrt{\frac{g}{\Delta_j}} \quad (4-7)$$
$$\Delta_j = C_{fn}\frac{5wsL^4}{384EI_{eff}} \quad (4-2)$$</td><td>$$f_n = \frac{\pi}{2}\left[\frac{gE_S I_t}{wL^4}\right]^{0.5} \quad (3.1)$$</td></tr>
<tr><td colspan="3">These expressions are almost identical, apart from the ATC method using US Customary Units and having them built-in to the expression. The AISC method uses a transformed moment of intertia while the ATC method uses a transformed stiffness. The coefficient C_{fn} is for joist continuity over the support. It is not explicitly included in (3.1), but the AISC also includes a similar expression. Since C_{fn} is taken as 1.0 there is no need to investigate this further. Similarly, I_{eff} is the effective moment of inertia, which a reduction of the gross moment of inertia due to shear deformation.</td></tr>
<tr><td>Expression for effective stiffness:</td><td>$$EI_{eff} = \frac{EI}{1+\frac{\gamma EI}{C_{fn}EI_m}} \quad (4-2a)$$</td><td>$$I_{eff} = \frac{I_{comp}}{1+\frac{0.15I_{comp}}{I_{chords}}} \quad (3.13)$$</td></tr>
<tr><td colspan="3">The expression in (3.13) is the same as (4-2a) if $C_{fn} = 1$, and $\gamma = 0.15$. Equation (4-2a) is recommended for simply-supported trusses. Equation (3.13) is a more general expression where g can be applied for various types of joists, and 0.0 is recommended for cold-formed C-joists. Shear deformation depends on the stiffness of the joist web. Because cold-formed C-joists have very thin webs, with web openings at discrete intervals, some shear deflection seems reasonable. The ATC guide has an expression for γ to be used for hot-rolled members that can be applied to cold-formed C-joists.</td></tr>
<tr><td>Expression for γ for hot-rolled members:</td><td>$$\gamma = \frac{9.6}{\left(\frac{L}{r}\right)^2}\frac{E}{G}\frac{A_m}{A_{web}} \quad (4-2b)$$</td><td></td></tr>
<tr><td colspan="3">For all of the joists types used in this study, the average value was 0.025, with a maximum of 0.052. Equation (3.13) recommends 0.15 for truss members with no web at all, and Eq. (4-2b) is not specifically applicable to C-joists. The ATC guide recommends 0.0, but no reference is given, and this value is unconservative from a design perspective. If a value of 0.045 is used, it should be reasonable and conservative, and sufficient until specific research has been conducted to find an appropriate γ for cold-formed C-joists.</td></tr>
<tr><td>The recommended expression for effective stiffness becomes:</td><td colspan="2">$$EI_{eff} = \frac{EI}{1+\frac{0.045EI}{EI_m}} \quad (4-2a\_mod)$$</td></tr>
</table>

<table>
<tr><td colspan="3">Finally, the remaining difference in the ATC and AISC methods is composite action. The AISC method assumes that the moment of inertia is fully-composite, while the ATC method had an expression to be used for light-frame construction that considers slip between the subfloor and joist. The area of the subfloor is effectively reduced when the parallel axis theorem is applied.</td></tr>
<tr><td>Area reduction for subfloor, accounting for non-composite action:</td><td>$$EA_{top} = \frac{EA_{flr}}{1+\frac{10EA_{flr}}{S_{flr}{L_{flr}}^2}}$$ $$(4-3b)$$</td><td></td></tr>
<tr><td colspan="3">The coefficient, S_{flr}, is the slip modulus. It is a function of the joist and subfloor material, connection type, and spacing. A value of infinity would indicate a fully-composite section. Values are given for nailed and nailed & glued connections for wood joists and deck products. Glue appears to provide an increase of approximately 10,000. Because no appropriate value was given, S_{flr} was taken as the value for nailed and glued OSB.</td></tr>
<tr><td colspan="3">Design Procedure Step 2: Calculate Acceleration Response (f_n < 15 Hz)</td></tr>
<tr><td>Peak acceleration due to walking excitation:</td><td>$$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} \quad (2-3)$$</td><td>$$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} \quad (4.1)$$</td></tr>
<tr><td colspan="3">These expressions themselves are identical. The different values of acceleration response will be computed because f_n (above) and W are calculated differently, and different recommendations for β are made by the design guides. The recommended damping ratio from Table 4-9 will be incorporated as a modification to both design procedures.</td></tr>
<tr><td>Effective floor weight:</td><td>$W = wBL \ (4-10)$</td><td>$W = wB_jL \ (4.2)$</td></tr>
<tr><td colspan="3">The variable B is the effective panel width (of the joist panel for the AISC procedure where j is the subscript)</td></tr>
<tr><td>Effective joist panel width:</td><td>$$B = C\left(\frac{D_{perp}}{D_{par}}\right)^{0.25} L \quad (4-11)$$</td><td>$$B_j = C_j\left(\frac{D_s}{D_j}\right)^{0.25} L_j \quad (4.3a)$$</td></tr>
<tr><td colspan="3">The above expressions are limited to a maximum of 2/3 of the actual floor width. The continuity factor C is 2.0 for most cases, but 1.0 if the joist panel is parallel to an interior edge. D_{par} and D_j are both effective stiffness (per unit width) of the floor in the joist direction, based on the transformed moment of inertia calculated in the previous step. D_{perp} and D_s are both effective stiffness (per unit width) of the floor in the transverse direction. The AISC design guide uses an average rectangular section for D_s, while the ATC design guide uses EI_{bi} (Eq. 4-5g), from the N_{eff} calculation.</td></tr>
<tr><td colspan="3">Design Procedure Step 3: Calculate Static Deflection from 1 kN Point Load</td></tr>
<tr><td>Expression for static deflection due to a 1 kN point load:</td><td>$$\Delta_p = \frac{C_{pd}}{N_{eff}} \frac{PL^3}{48EI_{eff}} \quad (4-1)$$</td><td></td></tr>
<tr><td colspan="3">There is no expression for deflection from a 1 kN point load in the AISC design guide, however, both guides provide a method for calculating N_{eff}. There are expressions for N_{eff} in both design guides. The load P is 225 lbs to correspond to 1 kN.</td></tr>
</table>

<table>
<tr><td>Effective flexural stiffness (static) of joist panel:</td><td>$EI_{eff} = \frac{EI}{1 + \frac{\gamma EI}{C_{pd} EI_m}} \quad (4-1\text{a})$</td><td></td></tr>
<tr><td colspan="3">The coefficient C_{pd} is 1.0 for simply supported joists; therefore EI_{eff} is the same is calculated for the dynamic stiffness, which is used to determine fundamental frequency.</td></tr>
<tr><td>Number of effective joists:</td><td>$N_{eff} = \frac{1}{DF_b - DF_v} \quad (4-6\text{a})$</td><td>$N_{eff}$
$= 0.49 + 34.2\frac{d_e}{S}$
$+ (9.00 \times 10^{-9})\frac{L_j^4}{I_t}$
$- 0.00059\left(\frac{L_j}{S}\right)^2 \quad (4.7)$

for $0.018 \leq \frac{d_e}{S} \leq 0.208$
$4.5 \times 10^6 \leq \frac{L_j^4}{I_t} \leq 257 \times 10^6$
and $2 \leq \frac{L_j}{S} \leq 30$</td></tr>
<tr><td colspan="3">In the ATC expression (4-6a), the variables DF_b and DF_v are empirical coefficients which are based on the bending (b) and shear (v) stiffness of the floor. For the AISC expression, there are three requirements that must me met in order for the calculation to be applicable. The variable de is the equivalent depth of the average rectangular cross section in the transverse direction.</td></tr>
</table>

Design Example for In Situ Floor CW805

This example is worked in Mathcad 14, which automatically converts units to the approprate system. Inputs are in US Customary in order to match the manufacturer's material data.

- Floor Dimensions:

$$L := 5.88m = 19.291 \cdot ft$$

$$B_{max} := 8.128m = 26.667 \cdot ft \text{ (total width)}$$

$$S := 2ft \qquad \text{(joist spacing)}$$

- Joist Properties: TDW16 (as provided by P1.11 of Dietrich *Steel Joist Design Guide*)

$$E_s := 29.5 \cdot 10^6 \frac{lbf}{in^2}$$

$$h := 12in \quad \text{(joist depth)}$$

$$t_{joist} := 0.0566in$$

$$A_{joist} := 0.953in^2$$

$$w_{joist} := 3.118 \frac{lbf}{ft}$$

$$I_{joist} := 17.677in^4$$

$$r := 4.478in$$

- Ceiling Properties: Drop Ceiling, 1 layer of Sheetrock

$$w_{ceiling} := 0.02 \frac{lbf}{in^2}$$

- Subfloor Properties: 1.5" LEVELROCK on 22 Ga UFS Deck
  (as provided by USG, and United Steel Deck)

$$f'_c := 3500 \quad \text{psi but non-dimensionalized for calculations}$$

$$t_{deck} := 0.0295in$$

$$w_{deck} := 1.50 \frac{lbf}{ft^2} = 0.01 \cdot \frac{lbf}{in^2}$$

joist direction — transverse direction

$$I_{deck\_J} := 0in^4 \qquad I_{deck\_T} := 0.025in^4$$

$$E_c := 1.35 \cdot 57000 \sqrt{f'_c \frac{lbf}{in^2}} = 4.552 \times 10^6 \cdot \frac{lbf}{in^2}$$ (1.35*empirical Ec expresison)

$$w_c := 120 \frac{lbf}{ft^3}$$

joist direction

$$t_{c\_J} := 1.0in$$

transverse direction

$$t_{c\_T} := 1.25in$$ (average)

Design Step 1: Compute Fundamental Frequency (Using Modified ATC Method)

$$EA_{flr} := E_c \cdot S \cdot t_{c\_J} = 1.093 \times 10^8 \cdot lbf$$

$$S_{flr} := 50000 \frac{\left(\frac{lbf}{in}\right)}{in}$$ (Nailed OSB, Table 4-3)

$$EA_{top} := \frac{EA_{flr}}{1 + \frac{10 \cdot EA_{flr}}{S_{flr} \cdot L^2}} = 7.761 \times 10^7 \cdot lbf$$ (Eq. 4-3b)

$$h_{top} := \frac{h}{2} + 0.5in + \frac{t_{c\_J}}{2} = 7 \cdot in$$ (distance from centroid of floor to centroid of joist)

$$y := \frac{EA_{top} \cdot h_{top}}{E_s \cdot A_{joist} + EA_{top}} = 5.139 \cdot in$$ (Eq. 4-3c)

$$EI_{top} := E_c \cdot S \cdot \frac{t_{c\_J}^{\,3}}{12} = 9.105 \times 10^6 \cdot lbf \cdot in^2$$ (Eq. 4-3a, no contribution from deck in this direction)

$$EI := E_s \cdot I_{joist} + EI_{top} + E_s \cdot A_{joist} \cdot y^2 + EA_{top} \cdot \left(h_{top} - y\right)^2 = 1.542 \times 10^9 \cdot lbf \cdot in^2$$

(Eq. 4-3)

$\gamma := 0.045$ (assumed, see attached notes)

$C_{fn} := 1$

$$EI_{eff} := \frac{EI}{1 + \frac{\gamma \cdot EI}{C_{fn} \cdot (E_s \cdot I_{joist})}} = 1.361 \times 10^{9} \cdot lbf \cdot in^{2}$$ (Eq. 4-2a)

$$w := 6\frac{lbf}{ft^2} + w_c \cdot \frac{(t_{c\_J} + t_{c\_T})}{2} + w_{ceiling} + w_{deck} + \frac{w_{joist}}{S} = 0.161 \cdot \frac{lbf}{in^2}$$

(includes 6 psf service load per ATC)

$$\Delta_j := C_{fn} \cdot \frac{5w \cdot S \cdot L^4}{384EI_{eff}} = 0.106 \cdot in$$ (Eq. 4-2)

$$f_n := 0.18 \cdot \sqrt{\frac{g}{\Delta_j}} = 10.853\frac{1}{s}$$ (Eq. 4-7)

Fundamental frequency is less than 15 Hz so both deflection and acceleration must be checked.

Design Step 2: Check Static Defleciton from 1 kN Load (Using Modified ATC Method)

This step is undertaken first because computed values are then incorporated in the calculation of the RMS acceleration response.

$$K_j := \frac{EI_{eff}}{L^3} = 109.687 \cdot \frac{lbf}{in}$$ (Eq. 4-5c)

$$EI_{wperp} := E_s \cdot I_{deck\_T} = 7.375 \times 10^{5} \cdot lbf \cdot in^{2}$$

$$EI_c := E_c \cdot \frac{S \cdot t_{c\_T}^{3}}{12} = 1.778 \times 10^{7} \cdot lbf \cdot in^{2}$$

$EA_{wperp} := 0 \cdot lbf$ no published area for steel deck, but very thin

$$EA_c := E_c \cdot S \cdot t_{c\_T} = 1.366 \times 10^{8} \cdot lbf$$

$$h_{cw} := \frac{t_{c\_T}}{2}$$

$$EI_{b1} := EI_{wperp} + EI_c + \frac{EA_{wperp} \cdot EA_c \cdot h_{cw}^{3}}{EA_{wperp} + EA_c} = 1.852 \times 10^{7} \cdot lbf \cdot in^{2}$$ (Eq. 4-5g) perpendicular flexural stiffness of floor panel

$$K_{b1} := 0.585 \frac{EI_{b1}}{S} \cdot \frac{L}{S^3} = 7.56 \times 10^{3} \cdot \frac{lbf}{in}$$ (Eq. 4-5d) stiffness of transverse flexural components (panel or deck components, strongback, solid blocking) - deck only

$$K_1 := \frac{K_j}{K_j + K_{b1}} = 0.014$$ (Eq.4-5a)

$$K_{v1} := 0 \frac{lbf}{in}$$ (Eq. 4-5f) stiffness of transverse shear components, provisions for solid blocking only

$$K_2 := \frac{K_{v1}}{K_{b1}} = 0$$ (Eq. 4.5b)

Note: improvments to this procedure can be made by developing a method of including the flexural contribution of the discrete blocking and ceiling, as well as shear contributions from discrete blocking and the subfloor. An alternative would be to develop an entirely new expression to compute Neff.

$$DF_b := 0.0294 + 0.536 \cdot K_1^{0.25} + 0.516 \cdot K_1^{0.5} - 0.31 \cdot K_1^{0.75} = 0.264$$ (Eq. 4-6b)

$$DF_v := -0.00253 - 0.0854 K_1^{0.25} + 0.0797 \cdot K_2^{0.5} - 0.00327 K_2 = -0.032$$ (Eq. 4-6c)

$$N_{eff} := \frac{1}{DF_b - DF_v} = 3.382$$ (Eq. 4-6a)

$$C_{pd} := 1$$

$$P := 225 lbf$$

$$\Delta_p := \frac{C_{pd}}{N_{eff}} \frac{P \cdot L^3}{48 EI_{eff}} = 0.013 \cdot in$$ (Eq. 4-1) deflection due to 1 kN point load

$$\Delta_{max} := 0.024 + 0.1 e^{-0.18\left(\frac{L}{1ft} - 6.4\right)} = 0.034$$ (Eq. 2-1)

Estimated deflection is significantly less than allowable so design is satisfactory based on the deflection check.

Design Step 3: Check Peak Walking Acceleration (Using Modified ATC Method)

$C := 2.0$ Table 4-6 for steel joist panel

$$D_{par} := \frac{EI_{eff}}{S} = 5.67 \times 10^{7} \cdot \frac{lbf \cdot in^{2}}{in}$$

$$D_{perp} := \frac{EI_{b1}}{S} = 7.717 \times 10^{5} \cdot \frac{lbf \cdot in^{2}}{in}$$

$$B := C \cdot \left(\frac{D_{perp}}{D_{par}}\right)^{.25} \cdot L = 13.178 \cdot ft$$ (Eq. 4-11) but less than $\frac{2}{3} B_{max} = 17.778 \cdot ft$

$W := w \cdot B \cdot L = 5.895 \times 10^{3} \cdot lbf$ (Eq. 4-10)

$P_o := 65lbf$ Table 2-1

$\beta := 0.045$ Table 4-9 (Parnell, 2008) for UFS sublfoor, ceiling, minimal partitions, no SRB

$$a_p := \frac{P_o \cdot e^{-0.35 f_n \cdot 1s}}{\beta \cdot W} = 0.549 \cdot \%$$ (Eq. 2-3)

This value is within the limit of 0.4 to 0.7% for residential occupancy, presented in Table 2-3, but alternatively the modified procedure can be applied for frequencies above 8 Hz.

$$a_w := \left(\frac{8}{f_n \cdot s}\right) a_p = 0.405 \cdot \%$$ (Eq. 5-1, Thesis)

This frequency weighted acceleration is less than 0.5%g, the limit from ISO 2631, so design is satisfactory based on acceleration check.

All checks are satisfactory based on proposed design method using modified ATC procedure.

For illustrative purposes, the AISC procedure will be followed, with similar modifications where applicable. This procedure is not recommended by this Thesis.

Design Step 1: Compute Fundamental Frequency (Using AISC Method)

$$EA_{flr} := E_c \cdot S \cdot t_{c\_J} = 1.093 \times 10^8 \cdot lbf$$

$$S_{flr} := 999999999999 \frac{\left(\frac{lbf}{in}\right)}{in}$$ (Infinitely large - fully composite)

$$EA_{top} := \frac{EA_{flr}}{1 + \frac{10 \cdot EA_{flr}}{S_{flr} \cdot L^2}} = 1.093 \times 10^8 \cdot lbf$$ (Eq. 4-3b)

$$h_{top} := \frac{h}{2} + 0.5in + \frac{t_{c\_J}}{2} = 7 \cdot in$$ (distance from centroid of floor to centroid of joist)

$$y := \frac{EA_{top} \cdot h_{top}}{E_s \cdot A_{joist} + EA_{top}} = 5.567 \cdot in$$ (Eq. 4-3c)

$$EI_{top} := E_c \cdot S \cdot \frac{t_{c\_J}^{\ 3}}{12} = 9.105 \times 10^6 \cdot lbf \cdot in^2$$ (Eq. 4-3a, no contribution from deck in this direction)

$$EI := E_s \cdot I_{joist} + EI_{top} + E_s \cdot A_{joist} \cdot y^2 + EA_{top} \cdot \left(h_{top} - y\right)^2 = 1.626 \times 10^9 \cdot lbf \cdot in^2$$
(Eq. 4-3)

$$\gamma := 0.15$$ (as specified by AISC for steel trusses)

$$C_{fn} := 1$$

$$EI_{eff} := \frac{EI}{1 + \frac{\gamma \cdot EI}{C_{fn} \cdot \left(E_s \cdot I_{joist}\right)}} = 1.108 \times 10^9 \cdot lbf \cdot in^2$$ (Eq. 4-2a)

$$w := 6\,\frac{\text{lbf}}{\text{ft}^2} + w_c \cdot \frac{\left(t_{c\_J} + t_{c\_T}\right)}{2} + w_{ceiling} + w_{deck} + \frac{w_{joist}}{S} = 0.161 \cdot \frac{\text{lbf}}{\text{in}^2}$$

(includes 6 psf service load per AISC)

$$\Delta_j := C_{fn} \cdot \frac{5w \cdot S \cdot L^4}{384EI_{eff}} = 0.13 \cdot \text{in}$$ (Eq. 4-2)

$$f_n := 0.18 \cdot \sqrt{\frac{g}{\Delta_j}} = 9.793\,\frac{1}{s}$$

Fundamental frequency is less than 15 Hz so both deflection and acceleration must be checked.

Design Step 2: Check Static Defleciton from 1 kN Load (Using AISC Method)

Note: there is no provision for calculating static deflection due to a point load. Neff will be calculated for comparative purposes.

$$d_e := t_{c\_T} = 0.032\,\text{m}$$

$$I_t := \frac{EI_{eff}}{E_s} = 1.563 \times 10^{-5}\,\text{m}^4$$

$$N_{eff} := 0.49 + 34.2\,\frac{d_e}{S} + 9 \cdot 10^{-9}\,\frac{L^4}{I_t} - 0.00059\left(\frac{L}{S}\right)^2 = 2.905$$ (Eq. 4.7)

Checks: $\frac{d_e}{S} = 0.052$

$$\frac{L^4}{I_t} = 7.647 \times 10^7$$

$$\frac{L}{S} = 9.646$$

All 3 satisfied

Design Step 3: Check Peak Walking Acceleration (Using AISC Method)

$C_j := 2.0$ for joists or beams in most areas

$D_j := \frac{I_t}{S} = 2.564 \times 10^4 \cdot mm^3$

$D_s := \frac{d_e^{\,3}}{12} = 2.667 \times 10^3 \cdot mm^3$ no contribution from form deck

$B := C_j \cdot \left(\frac{D_s}{D_j}\right)^{.25} \cdot L = 6.678 \cdot m$ (Eq. 4.3a) but less than $\frac{2}{3} B_{max} = 5.419 \cdot m$

$W := w \cdot B_{max} \cdot L = 53.064 \cdot kN$ (Eq. 4.2)

$P_o := 65lbf$ Table 4.1

$\beta := 0.045$ Table 4-9 (Parnell, 2008) for UFS sublfoor, ceiling, minimal partitions, no SRB

$$a_p := \frac{P_o \cdot e^{-0.35 f_n \cdot 1s}}{\beta \cdot W} = 0.393 \cdot \%$$ (Eq. 4.1)

This value is within the limit of 0.4 to 0.7% for residential occupancy, presented in Table 2-3, but alternatively the modified procedure can be applied for frequencies above 8 Hz.

$$a_w := \left(\frac{8}{f_n \cdot s}\right) a_p = 0.321 \cdot \%$$ (Eq. 5-1, Thesis)

This frequency weighted acceleration is less than 0.5%g, the limit from ISO 2631, so design is satisfactory based on acceleration check.