FUNDAMENTAL FREQUENCIES OF I-JOIST, SOLID-SAWN WOOD JOIST, AND TRUSS FLOORS BASED ON TEE-BEAM MODELING

by

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(ABSTRACT)

Full size wood joist floors were built and their frequencies were measured. Double tee-beam floors were built and also cut from the full size floors and their frequencies were measured. The floor joists investigated included: solid sawn, parallel chord floor trusses, and composite I-joists. The first natural frequency of each tee-beam floor was predicted after measuring the deflection under imposed loading. A design procedure is presented to predict the fundamental frequency of wood joist floors from tabulated material properties.

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CHAPTER I

INTRODUCTION

With the advent of new building materials such as composite wood I-joists and the increased use of parallel chord floor wood trusses the reality exists that wood floors can span distances previously not possible. As with all building materials, the design engineers main concern (along with that of the public) is strength. 'Is the structure strong enough to support the anticipated loads without structural failure', this has been, and is, the first check made by the design engineer, and is often the governing factor in building design. However, when structural systems have a lighter self-weight and beam spans are longer, annoying floor motion caused by human activity can be a major concern to the building occupants.

1.1 Background

Since a major redeeming quality of wooden floor systems is the high strength to weight ratio of the material, annoying floor vibrations of wooden structures can be a problem. Usually, after the floor system is designed on the strength requirements, serviceability of the floor is checked based on the deflection for a uniform static load. These two limit states, strength and deflection serviceability, do not provide enough information to allow the designer to predict if the floor will produce annoying floor motion to the building occupants.

If a floor system produces motion that is of significant annoyance to the occupants, then the system can be labeled as a 'structural failure'. A satisfactory structural design is based on two criteria: strength and serviceability. When a structure fails the criterion of vibrational serviceability the structure is by definition a structural failure even though the building is not in danger of collapse. The problem for design engineers is that for a long time design criteria for vibrational serviceability did not exist and when such criteria were developed they either did not give correct results and/or were so complicated that design engineers had difficulty applying them. Consequently, there is need for a vibrational criterion which is simple to use, dependent only on information known at the design stage, incorporates vibrational parameters, and is based on a human perceptibility scale.

During the past twenty years much work has been done in the area of wood floor vibrational research, though mainly in Canada and Sweden. Many design criteria have been proposed, but to date there has been no criterion that is truly designer friendly and at the same time encompassing all parameters that affect human perception of floor vibration. One or more parameters, such as amplitude, frequency, or damping, are often left out of the criteria. Some criteria are even based on a maximum deflection value due to a static load. An in-depth study of past research is presented in the next section.

1.2 Literature Review

1.2.1 Introduction

This literature review will present work previously conducted in the area of floor serviceability. The review will concentrate on the vibrational aspect of serviceability as associated with wood floors. Each author's work will be presented and discussed completely before moving onto another researcher's work. In this section, the word author(s) refers to the individual who's work is being reviewed, and not to the writer of this study.

This writer, after thoroughly investigating the previous research conducted in the area of wood floor vibration, determined that the work of five authors: Polensek, Foschi, Onysko, Chui, and Ohlsson are the most beneficial to this research. Therefore, an in-depth investigation of these authors is presented. Also, the work of a number of other researchers is included, but the discussion is limited to the factors that affect the present work.

1.2.2 Investigation of Five Major Authors and Their Research Into the Problems of Vibration of Wood Floor Systems

Polensek [1970] is one of the earliest papers addressing the topic of occupant response to vibration of wood floors. He stated that the first thing that should be established is a subjective rating scale to determine acceptable levels of vibration by occupants. He stated that human tolerance to vibration depends on the frequency of the vibration. "For instance, exposure for 1 minute to sinusoidal vibration with frequency between 4 and 6 Hz, with an acceleration of 1 g (unit of acceleration because of gravity), is about the limit that humans can tolerate. Similarly, the tolerance level at 10 Hz is about 2 g, and at 20 Hz, about 4 g.". Polensek interestingly commented that "the physical nature of sound and structural vibration are the same, so proper acoustical design can also diminish objectionable vibration of the floor system."

A person walking on a floor system will induce low amplitude vibrations that can be more objectionable to the occupants than a vibration induced by an sudden impact force of much higher amplitude. The magnitudes of the impacts depend on the characteristics of the person walking and not so much on their weight. He cited studies that have shown that heavy men usually walk with less impact that lighter men and people that are carrying something walk with less impact then when not carrying something.

Polensek derived a frequency equation that is basically an eigen-value problem solution. He wrote a complex program to solve his equation. The predicted frequencies are within 2% of experimental test values. His experiments consisted of building 28 wood joist floor systems of varying dimensions. The flooring consisted of one layer of 1/2 in.

plywood; nails were used for connections. The test frame consisted of a 9 in. thick 4 ft. high concrete wall. The floors were tested by dropping a 70 lb. steel weight onto a 1-1/2 in. thick felt pad, located at the center of the floor. LVDT's were used to measure resulting floor displacement versus time. The floors were also measured subjectively by 'professionals'. These 'professionals' consisted of informed individuals associated with the research who sat in uncushioned chairs at midspan.

Over 1,200 vibration traces were collected over a two year period. He graphed joist deflection versus frequency and then overplotted the results of the subjective tests. The overall result was: the larger the amplitude the more objectionable the vibration no matter what the frequency.

Polensek [1971] again attempted to evaluate the vibrational properties of wood joists. This time he used connections of glue and nails. The whole purpose of this research was to evaluate what effect elastomeric adhesive (glue) had on floor performance. Testing performed on the floors to evaluate vibrational characteristics included: vertical free-vibration, horizontal vibration, human response to vibration, and vertical vibration with support restraints and added mass tests.

The results of the tests produced many interesting points. One of which was that the glue had no apparent effect on the range of basic modal frequencies which were expected to increase due to the glue. Polensek said it was purely coincident and offered this theory:

> "In dynamic analysis, the floor can be considered as a system of elastically supported masses concentrated in joists. Each individual mass has its frequency denoted here as the primary frequency, which is higher for glued joists than for their nailed counterparts. The floor covering acting in a direction perpendicular to the floor span elastically connects these masses. As a result, several frequencies different from the primary frequencies, act at each joist. How the primary frequencies combine depends very much on their relative values and not only on their absolute magnitudes. Therefore,

> smaller primary frequencies of a nailed floor can combine into larger final frequencies than somewhat larger primary frequencies of the glued floor."

Other results indicated that damping capacity of the floors was not greatly changed by the use of glue. However the gluing made the floors more acceptable to human response with regard to both impact and walking vibration. This is because the human response to floor vibration is greatly influenced by maximum peak-to-peak deflection and less by frequency (Polensek stated this in his 1970 paper). Polensek also stated that the addition of mass to the floor did not change the damping capacity but it decreased the fundamental modal frequency. Restraining of the ends of the floor had no effect on damping and only slightly increased the natural frequency.

Polensek [1975] then decided to pursue the idea of damping. He investigated the damping capacity of wood floors connected by nails only. He stated that vibrations that are objectionable can be eliminated by increasing the stiffness of a floor or its damping capacity. He used data and floors from his previous research in this study. Floors were set in free vibration by suddenly removing a 200 lb. weight positioned at the center of the floor. He showed that at large amplitudes, slip occurs in the nails between sheathing and joists. The slip produces fast decay of these large amplitudes. Friction and energy dissipation from this tend to increase damping. At smaller amplitudes enough shear capacity exists in the connection to resist slip. Therefore, these smaller amplitudes decayed more slowly. Polensek used viscous damping in analyzing the traces (viscous damping assumes damping force is proportional to velocity and opposite to the direction of motion). Damping was estimated by the logarithmic decrement method.

The author performed experiments on the floors to determine the damping capacity provided by humans. It was found that the human body increased damping of a floor system. Also it was found that the weight and physique of the human had a measurable effect upon damping.

Polensek et al. [1976] by further analysis of Polensek's previous studies and the results of additional study, determined that amplitude was the best single factor in Polensek's human response scale. However, this time they stated that human perception of vibration is affected by more than one variable. They suggested such factors as damping and frequency in addition to amplitude. They did stand firm on his statement that "static and geometric properties of wood joist floors are not good indicators of human response".

Polensek [1988] summarized the results of much of his work on the effect of damping on human response to floor vibrations. The paper contains a wealth of information on damping as related to wood joist floor systems and the reader who is interested in this topic is highly recommended to review this paper by Polensek.

He stated that "damping in wood floor systems is caused by friction originating internally in the wood and in the contact interlayers between the various components." He divided damping into internal material damping and mechanical damping. Polensek stated that no values of damping exist for such things as furniture, carpet, or other such objects. He reviewed the Rayleigh matrix relationship of damping:

$$[D] = \alpha[M] + \beta[K] \tag{1.1}$$

where [D] = displacement matrix,

[M] = mass matrix, and

[K] = stiffness matrix.

and α and β are scalar constants defining the fraction of mass and stiffness used in damping. Polensek stated this principle is fine for steel and concrete floor systems, but gives poor results for floor systems that contain materials with highly variable properties.

Polensek said wood floors typically have damping ratios between 5% and 10% of critical. The joints between sheathing and joist provide the main source of damping. He therefore modified Equation 1.1 to account for this source of damping.

Foschi [1985] found, while researching how to predict first joist failure loads using 13 wood floors he constructed, that static floor behavior was linear up to the failure of the first joist. This is important since it proves that linear structural analysis procedures are applicable for floor vibration, since the load is well below the load that at which failure will occur.

Foschi and Gupta [1987] performed a study of the reliability of floors under impact vibration. In this paper they showed results of their analysis of vibrations of wood floors using a finite strip analysis. The original program was developed for static loads but the authors extended it to solve for dynamic problems.

The authors made a comparison between the results of their finite strip method to actual human reactions to the floors. They then proposed a simple method (based on statics) that a designer could use to determine if the proposed floor system would produce perceptible vibrations. If a 1 kN load placed at midspan of the bare joist produced less than 1 mm of deflection, then the floor system would be acceptable by the human criteria used (this is equivalent to a stiffness of 5,625 lb./ft.). The human criterion used for this criteria was that proposed by Wiss and Parmelee [1974].

Foschi and Folz [1988] performed reliability tests of wood floors. They used three criteria in this research, two of them come from recommendations of a consumer report study while the third is based on dynamic analysis combined with human acceptance criterion. The authors stated that the then current deflection requirement of L/360 for a uniform load of 40 psf is inadequate to prevent vibrational problems.

In their reliability analysis they considered the MOE of the joists to be a random variable and the reliability index of a floor span was computed by First Order Methods.

They proposed a design equation that complied with all their criteria in their reliability analysis. The equation is simple and predicts floor vibrational acceptability, based on static deflection. Where K will represent the maximum permissible floor deflection, for vibrational acceptance.

$$\frac{5}{384}\frac{wsL_o^4}{EI}=\frac{L_o}{K} \qquad (1.2)$$

where K = required deflection limitation,

L_o = joist span, from reliability analysis,

EI = mean stiffness of the joists',

s = joist spacing, and

w = 40 psf distributed load.

D. M. Onysko also proposed a simple design equation [1988]. The motivation of his work was the fact that the then current deflection requirement of wood floors proved to be inadequate to meet all serviceability problems (vibrations). Onysko's proposed criterion is

$$y=\frac{7.217}{L^{1.274}} \qquad (1.3)$$

where L = clear span (meters), and

y = deflection (millimeters).

If y is less than the deflection produced by a 1kN load placed at midspan then the floor will be acceptable. Equation 1.3 was simplified by Onysko, after he reevaluated his criterion based on more recent in-grade data available for materials

$$y=\frac{8.0}{L^{1.3}} \quad \text{(mm)} \qquad (1.4)$$

The basis for equation 1.3 is explained in Onysko [1975]. In the early 1970's, he conducted surveys of occupants of wood joist floor buildings. He made up a questionnaire, filled out by occupants, that contained a vast assortment of questions concerning housing type, city, and different characteristics of vibrations felt by occupants. After the occupants filled out the questionnaires, Onysko's research staff performed a complete inspection of the floors (measurements, materials used, etc.). From all this information a computer program was written to plot the data gathered. On this plot a curved line was drawn to separate acceptable and nonacceptable floors Equation 1.3 represents this curve. The shape of this curve is only slightly affected by the modifications for Equation 1.4. Onysko states that one method to avoid annoying floor vibrations is dependent "on the quality of the data and on the number and nature of the factors that are controllable in design."

From his field studies, Onysko found that the most important variable was not deflection of floors under uniform load but a combination of the following: deflection under concentrated load, floor span, amplitude-to-impulse loading, damping, and natural frequency.

Concluding the work of the previous two authors, (Foschi and Onysko both of whom worked independently from one another), the concept of human response to floor systems entered heavily into both of their investigations. Both investigators also realized the need for an easy to use design equation that would allow engineers to determine if a floor would be acceptable in the design stage. The equations both authors proposed are based on a static deflection.

Chui [1986] stated that damping is one of the most important variables affecting floor vibration. When damping is properly utilized unsatisfactory vibrations can usually be eliminated. Since wood floors are complex structurally, few design oriented and simple equations have been developed for estimating vibrational behavior of wood floors. Chui's objective of this research was to determine what type of construction techniques have an effect on the vibrational characteristics of wood floors.

Heel drops and/or hammer impacts were used to excite the test floors. The impact force was measured by a load cell while the floor response was measured by an accelerometer. Chui observed that modes of vibration of a wood floor are closely spaced. This is because wood floors are highly orthotropic. Modes should be kept wide apart or else they will interact and produce vibration with high amplitude. He proved that reducing joist spacing (consequently modal distance will decrease) will have a negative effect on vibrational performance. He also reviewed how blocking, bridging, and other construction factors, when properly installed, affect floor performance. He stated that plywood is better than chipboard which is in-turn better than softwood sheathing for floor performance.

Within the limits of working loads, the end conditions (pinned-pinned versus fixed-fixed) had very little effect on floor performance. However, when the floor was supported on all four sides, instead of only two sides, and an internal support was added, the natural frequency rose by nearly 20% (due to stiffening of the floor system).

Chui stated that the basic requirement of a design equation is that it must cover all the contributing variables of floor vibration. Human response to floor vibrations is dependent on three things: amplitude, damping, and frequency. He found that "the root-mean-square (RMS) of the frequency-weighted acceleration of a transient vibration caused by a walking impact satisfies this requirement."

$$A_r = \sqrt{\frac{1}{T}\int_0^T a^2(t)\,dt} \tag{1.5}$$

where T = vibrational duration (seconds), and

a(t) = acceleration at time t seconds

Only a small number of in-situ floors were tested and analyzed by the above equation, but it was found that for floors with RMS accelerations less than 0.375 m/s^2 (or 3.8% g) human perception of the floors were good.

Smith and Chui [1988] proposed a design procedure with the following steps: (1) Make sure that the floor satisfies static design codes for strength and deflection. (2) Calculate the fundamental natural frequency. (3) If frequency is greater than 8 Hz, continue; if not, go back to step (1) and redesign so frequency is greater than 8 Hz. (4) Estimate RMS acceleration. (5) If RMS acceleration is less than 0.45 m/s^2 (or 4.6% g) design is concluded, if not modify design, go back to step (1).

The authors mathematically developed an equation for a designer to use to estimate RMS acceleration. Chui [1988] stated: "In the structural sense a wooden floor can be treated as a thin plate reinforced by a series of ribs. Some researchers simplified it further by regarding it as an isotropic or orthotropic plate. A design method based on this approach is difficult to practice as the 'equivalent' floor stiffnesses in the orthogonal directions cannot be evaluated easily."

Chui proposed that the fundamental natural frequency, f_n, of a wood floor be calculated using:

$$f_n = \frac{\pi}{2a^2}\sqrt{\frac{EI(N_j - 1)}{\rho_f bh + \rho_j A(N_j - 1)}} \quad \text{(Hz)} \qquad (1.6)$$

where

EI = mean joist stiffness,

N_j = total number of joists,

a = span of floor,

b = width of floor,

h = flooring thickness,

A = cross sectional area of joists,

ρ_f = density of flooring, and

ρ_j = mean density of joists.

The above equation is for a simply supported (on four sides) wood floor and is based on Rayleigh's principle and does not account for composite action between sheathing and joists. A method of taking into account composite action was discussed in Smith and Chui [1988].

Their simplified method of calculating RMS acceleration (A_r) is shown below (Chui derived this by using the Duhamel integral):

$$A_r = \frac{2000}{m\pi f_o} K \qquad (m/s^2) \tag{1.7}$$

where

$$m = \frac{a}{2}\left[\rho_f hb + \rho_j A(N_j - 1) + \frac{280}{a}\right] \tag{1.8}$$

$$f_o = \frac{\pi}{2a^2}\sqrt{\frac{EI(N_j - 1)}{\rho_f hb + \rho_j A(N_j - 1) + \frac{280}{a}}} \tag{1.9}$$

and,

$$K = \mathrm{f}(\gamma, f_o, \mathrm{t}_1) = \sqrt{\frac{\omega_n\left[1 - e^{-2\gamma\omega_n t_1} + \left(1 - e^{-2\gamma\omega_n(1-t_1)}\right)\left(1 - 2e^{-\gamma\omega_n t_1}\cos\omega_n \mathrm{t}_1 + e^{-2\gamma\omega_n t_1}\right)\right]}{4\gamma}} \tag{1.10}$$

K is a function of the viscous damping ratio γ, the fundamental natural frequency f_o, and the duration of the impact t_1. One can calculate K, but Chui presented a table of K values for fundamental frequencies (f_0) of 8 Hz to 40 Hz, which range from 38.8 to 80.2, respectively. These equations assume that the floor acts as a single degree-of-freedom system. This should be a reasonable assumption if all four edges of the floor are supported, and bridging is adequate and properly installed. Chui warned that when inadequate bridging is used a more complex model of the floor is required. Chui established a guideline of how many rows of bridging should be used per span length. However, Chui [1986] stated that in actual construction, bridging is usually improperly installed to such a degree that it has no effect upon vibrational performance of the floor system.

Chui and Hu [1990] investigated the dynamic response of composite wood I-joists. They state that human reaction to vibrations is influenced by frequency and mean level of vibration. These two conditions are both dependent on mass. That is why light structures, such as wood floors, tend to have vibrational problems. The analysis of the test floors determined that the I-joists had higher frequencies than their equivalent solid joist counterparts for identical spans. This is because the I-joists have a lower self-weight for a given stiffness. Although the frequencies of the I-joists were higher than their solid joist counterpart, their RMS acceleration values were all greater than Chui's acceptable limit of 0.45 m/s^2. An interesting note is that these I-joist floors had much higher damping than their solid joist counterparts.

Hu [1991] prepared a numerical model that was verified both numerically and experimentally. The conclusion from this work was "neglect of shear deformation and rotatory inertia in ribs can lead to an over estimate of the stiffness of a ribbed plate and to large errors in predicted frequencies and time history responses".

By the use of finite element method (FEM), Filiatrault et al. [1990] compared their values of frequency to the ones obtained by Smith and Chui's [1988] proposed design equation. The results obtained by the FEM show that Smith and Chui's approximate expression for frequency is inadequate in many circumstances. According to Filiatrault et al., Smith and Chui's equation under predicts the frequency by more than 40% in some cases.

Ohlsson [1986] stated that floor performance is dependent on floor mass. If mass is reduced in floor design, then it must be made up in stiffness. Footstep forces have dominate low-frequency components (up to 6 Hz), but at the same time they also have high-frequency components (6 Hz to 40 Hz). These high level frequencies have intensities which are almost inversely proportional to the frequency. Therefore, two different scenarios can result. (1) Floors with natural frequencies below approximately 8 Hz ("large force multiplied by low flexibility"). (2) Floors with fundamental frequencies well above the low-frequency region or 8 Hz ("small force multiplied by high flexibility"). Ohlsson's [1986] paper is concerned with scenario (2).

Ohlsson stated that the results of dynamic analyses have shown that joist bending rigidity governs the fundamental frequency, and that transverse and local rigidities affect the spacing of the higher frequencies and consequently the number of modes that contribute to the vibrational response of the floor. Damping is a good indicator of vibrational performance since it reduces amplitude and increases decay rate. Preliminary results using visco-elastic adhesive have shown promising results of increasing floor damping.

An interesting experiment was performed on an 'ideal' piece of lumber by Ohlsson and Perstorper [1991]. They suspended the test specimen from both ends by ropes to simulate pinned-pinned connections. They tested 13 different modes of vibration. The calculated modes of eigenfrequencies for higher modes of vibration were very close to the experimental values. The experiment confirmed that wood properties could be accurately estimated using modal testing and corresponding computer analysis.

Ohlsson [1988a] wrote a paper about vibrational research that he conducted over a ten year period. He made a good point that most people think of serviceability as a criterion based on static loads alone. However, vibrational problems (which are dynamic loads) usually are the critical load case. This led him to believe that vibrational, rather than strength requirements, may govern future design criteria.

Ohlsson divided floor vibrations between springiness and vibrational disturbances. He defined them as follows;

> -Springiness of a floor is associated with the sensation of self-generated floor deflection and vibration from a single footstep during the time of contact between foot and floor surface.
>
> -Vibrational disturbances caused by foot-fall on a floor are characterized by perception of floor vibration induced by persons other then the one that is perceiving it.

He stated that the problem of springiness is associated with light-weight floors and floors with low local rigidities as compared to the concentrated load applied to the floor. According to Ohlsson the response of floors with natural frequencies below 8 Hz will be totally dominated by the natural frequency. Therefore, the RMS acceleration will work well for predicting a problem floor vibration. If the preceding statement is true, then for floors with fundamental frequencies greater than 8 Hz the floor response will be a factor of many modes of frequency or even dependent on the second, third, or fourth harmonic. Most human senses are thought to posses a logarithmic, rather than a linear, character. If this is true then human perception of vibration is complicated.

Ohlsson calculated frequency from

$$f_n = \frac{\pi}{2}\sqrt{\frac{D_x}{gL^2}}\sqrt{1+\left(2n^2(L/B)^2 + n^4(L/B)^4\right)\frac{D_y}{D_x}} \qquad \text{(Hz)} \qquad (1.11)$$

where D_x and D_y are the plate bending rigidities ($D_x > D_y$),

g is mass per m^2,

L/B is the aspect ratio, and

n is the mode number.

The above equation is for orthotropic plates. According to Ohlsson, damping is probably the most misunderstood parameter of vibrational problems. Damping is important because it limits vibrational amplitude and increases vibrational decay rate.

In this paper, Ohlsson stated the difficulties associated with developing design equations to control floor vibration. He mentioned such things as with respect to amplitude and frequency the real dynamic loading is not known, and partitions and other non-structural components further complicate the matter, along with the relative damping of a system. Relative damping is a very important variable in controlling floor vibrations, and usually has to be guessed by the engineer. Also that human perception is dependent on the individual and the environment that the individual is in.

Ohlsson realized that with all these criteria one must consider when attempting to develop a design equation, the process can become overwhelming. He therefore stated that one should be "humble when discussing different proposed vibrational design criteria".

Ohlsson [1988b] also published a design guide that engineers can use to avoid vibrational problems in floors. This book was an extension of the results he obtained while working on his Ph.D. Although the design methodology in the book were developed for wood floors, the equations are completely independent of construction material and many examples of floors constructed with materials other than wood are presented.

Ohlsson [1988c] also presented a paper that reviews the design method presented in his book. This method is dependent on three variables; static flexibility, unit impulse velocity response, and steady velocity response. He stated that this guide has been in use in Sweden since 1984 and has produced acceptable floors. One fact that makes his method easier to use than other methods is that Ohlsson supports his method by numerous design aids, such as, diagrams, flow-charts, and worked examples.

It should first be noted that his method deals only with floors with natural frequencies that are greater than 8 Hz. To his knowledge no one knows which representation of a transient vibration gives the best measure of human disturbance, but he does note the following:

" - Short duration is very favorable. This means that high values for the floor structural damping is advantageous. - Different frequency components interact to produce annoyance. This means that considering one mode only will strongly underestimate the experienced discomfort."

His method makes use of, h'max, which is the initial value for the unit impulse vibration velocity response. This h'max is the design parameter and its limiting value is dependent on the expected damping coefficient. The value h'max is a structural property that, for a given loading point, is defined as;

$$h'\max = \sum_{n=1}^{\infty} \Phi_n^2(x_0, y_0)/m_n \quad ((m/s)/Ns) \qquad (1.12)$$

where x_0, and y_0 are the location,

n is the mode number,

$\Phi_n(x,y)$ is the mode shape function for mode number n, and

m_n is the modal mass for mode n.

In considering floor vibrations the value for ∞ is set at N_{40} which is the number of normal modes with natural frequencies less than 40 Hz. Next, the natural frequencies and corresponding modes and modal masses are calculated. Then the point that produces the largest value of h'max (based on x_0, y_0) is identified, and h'max is thereby calculated. This procedure is aided by Ohlsson's design charts. If approximations relating to modal masses are made then the h'max equation simplifies as shown below:

$$h'\max = (4/(gBL))\left(\sum_{n=1}^{N40} \Phi_n^2(L/2, y_0)\right) \qquad (1.13)$$

where g = weight per unit area, and

BL = plan dimensions.

Ohlsson found that the value of y_0 approaches zero for increasing number of modes. He thereby simplified his equation to

$$h'\max = 4(0.4 + 0.6N_{40})/(gBL) \qquad (1.14)$$

where N_{40} can be read off his design charts, and

gBL is the known mass of the floor.

He next included the dead weight of the non-vibrating portion of the human body (approx. 50 kg) to arrive at the final equation:

$$h'\max \approx 4(0.4 + 0.6N_{40})/(gBL + 200) \qquad (1.15)$$

He maked note that the above equation is proportional to the number of modes (below 40 Hz) and that the number N_{40} is dependent on the flexural stiffness perpendicular to span. He also noted that the value of h'max is approximately inversely proportional to the mass (gBL) of the floor. The next parameter Ohlsson calculated is the damping coefficient (σ_0) of the floor.

Ohlsson has a rating chart for floor performance which consists of a y-axis defined by h'max values (logarithmic) versus x-axis values of damping coefficients. The designer locates the point on the chart that he obtained by following the procedure outlined above. The chart is divided into three categories: intrusive, uncertain, and better.

Ohlsson also presented a method to design floors for steady state vibration. He found, like other researchers, that by providing a limiting value for the root-mean-square of the acceleration, unacceptable vibrations could be avoided. This equation is based on one walking person:

$$w'_{RMS} = \sqrt{\int_{8}^{40} S_{FF}(f)|M(f)|^2\,dx} \qquad (m/s)_{RMS} \qquad (1.16)$$

where S_{FF} (f) is the power spectral density of the generated force, and

M(f) is the mobility of the 'weakest point' of the floor.

He stated this type of vibration in 'short span' wood floors is seldom the governing factor of design

Ohlsson [1988d] commented upon Smith and Chui's proposed design method. He plainly stated "that a sound engineering development of floor structures will benefit if: The design proposal (Chui and Smith 1988) is not accepted for inclusion in the CIB model timber code." Ohlsson stated that a design method for floor vibration should;

" (I) Reflect the degree of vibrational serviceability. (II) Be neutral with regard to construction material and structural configuration. (III) Only include such well defined quantities, that are possible both to calculate and to measure. (IV) Be clearly understandable and enable the engineer to predict the effects of structural modifications."

Rule (II) above is violated by Chui and Smith's proposed criterion since they assume the fundamental mode alone is governing the floor response. According to Ohlsson this assumption will strongly underestimate the response of a transient loaded, lightweight, strongly orthotropic floor (wood floors). Chui and Smith's criterion also violates the criteria since it is dependent on type of construction and material. According to Ohlsson even the use of the heel drop violates rule (III).

Smith and Chui [1991] addressed the comments made by Ohlsson concerning their proposed design equation (1988). They basically defend their proposed design method and explain why it is dependent on certain properties of the floor. They indicate that the response parameters change as the ratio of imposed floor mass to self mass changes. This is why it will be extremely difficult to use the same design equation for concrete (low imposed mass to self mass ratio) and also for wood floors (high ratio). It is highly recommended that the interested reader should review this paper.

Subsequently, Smith and Chui built wood floors to collaborate their previous research while adding argument to their defense of Ohlsson's [1988d] charges. During this research they found that random variability of joist stiffness and mass diminishes with the increase of the number of floor joists in the system. They found it better to use joists of lower grade but larger cross section when vibrational design governed. Bridging and edge support helped lower and gave more uniform RMS accelerations (because of the increased effect of load sharing) thus improving floor performance.

To sum up their comments is to say that a design equation will have to be dependent on such factors as imposed mass, its distribution, and the excitation. These factors are all highly dependent on the type of floor being analyzed. This writer ends this section of the literature review with these parting comments of Smith and Chui;

> "Although a worthy goal to which to aspire, material and structural configuration neutral vibrational response criteria for joisted floors are not valid for contemporary engineering design codes."

1.2.3 Review of Related Literature

Hurst and Lezotte [1970] investigated the effect of joist size and flooring on the frequency and damping of wood floor systems. The test floors built consisted of nominal 2 in. x 4 in., 2 in. x 6 in., 2 in. x 8 in., and 2 in. x 10 in. joists. The flooring consisted of one layer of 1/2 in. plywood subflooring with 25/32 in. x 2.25 in. oak strip flooring laid parallel to the joists. The floors were also tested with the oak stripping laid perpendicular to the joists. In all floors, the joists were 16 in. center-to-center. The hardwood flooring was connected using 8d nails and the average moisture content of the joists was 11%. The floors were vibrated by applying a concentrated load and releasing it suddenly. The resulting vertical free vibration was measured with a linear variable differential transformer type displacement transducer (LVDT).

They stated that the frequency of a vibrating system is as follows:

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \qquad \text{(Hz)} \qquad (1.17)$$

where f = frequency,

K = the spring constant of the system, and

M = the mass of the system.

For the floors tested, it was determined that the floors could be modeled as a simply supported beam. The authors determined that the frequencies of the floors ranged between 8.1 Hz and 23.0 Hz. However, if they did not include the results of the 2 in. x 4 in. joist floors, since floor systems usually do not have joists smaller than 2 in. x 6 in., then the frequency range was 13.2 Hz to 23.0 Hz. The authors found damping, by the logarithmic decrement method, of the floors to range between 2.25 and 5.50 percent of critical.

The conclusions they drew from this research are as follows: The direction of the flooring (parallel vs. perpendicular) to the joist affected the frequency, this is because of the change in overall stiffness of the system. The direction of the flooring has no effect on the damping of the system. And, resonance conditions due to walking of the occupants, will not occur in residential wood floors.

Wheat and Moody [1984] used a finite element program called FEAFLO to predict the strength of wood joist floors. This program is based on a linear model and gives adequate prediction of floor stiffness at service loads but it does not adequately predict failure loads. The reason for this is that nailed joints do not exhibit linear behavior near failure loads. Two important points can be drawn from their research. The first is that the loss of stiffness at very high loads is due to loss of stiffness of one or more of the constituent parts of the floor such as joists, sheathing, or connectors. The second is that at very high loads creep produces a nonlinear load-deflection response. The useful conclusions of their research that are pertinent to this research are: wood floor connections exhibit linear behavior at service loads, and load versus deflection is linear for service loads.

Corder and Jordan [1975] constructed numerous floors all consisting of 2 in. x 8 in. joists. The floors contained three or four joists varying in length. The sheathing ranged from only particleboard flooring to particleboard subfloor in combination with plywood flooring. Both square edge and tongue-in-grove materials were used. Connections consisted of nails and/or glue. The joists were tested in edgewise bending to determine modulus of elasticity (MOE). Four test scenarios were conducted on each floor. The tests performed were: fixed line load, vibration, concentrated load, and impact tests.

Each vibration test was performed by suddenly releasing a 200 lb. load that was connected to the center of the floor panel and recording each joist movement by LVDT's. The impact test consisted of dropping a 150 lb. bag of lead shot on the floor from varying heights and measuring the instantaneous joist deflections.

The results of the vibration tests showed that the natural frequency of the test floors ranged between 14 Hz and 20 Hz. Damping ranged between 2.7 and 8.3 percent of critical. The floors that were connected only by nails, had higher damping values than the nailed and glued floors. This was attributed to 'slippage' between joists and flooring.

Impact test results showed that the floors using plywood sheathing only, deflected less that floors using particleboard sheathing only. This is because plywood is stronger and stiffer than particleboard.

The test results from this research indicated that the stiffness of a joist frame was increased almost 100% by the addition of plywood to a particleboard floor system (i.e. two layer sheathing).

McCutcheon et al. [1981] constructed eleven wood-joist floors using typical construction techniques. They measured the static deflection of the floors and then selected five of the eleven floors for more intensive study. The floors included single and double sheathed floors with nailed and/or glued connections. The nailed floors were designed not to exceed a deflection under 40 psf live load for span/360. The glued floors were designed considering composite action in calculating stiffness but the strength was computed on the basis of the bare joists. The floors contained eleven joists, rigid supports were beyond the first and eleventh joists.

The floors were modeled using a mathematical model developed at Colorado State University (CSU) and by the use the existing finite element program, FEAFLO. For uniformly loaded floors, the average midspan joist deflection appeared to be a good measure of floor performance, therefore the authors used this measurement to determine floor performance during the course of their research.

They used the value, k, to represent the assumed linear slip modulus between connector load and deformation. If sheathing and joist are unconnected, k is very low and no composite action is developed. However if rigid glue was used, almost 100% of the potential composite action develops. They found that the nailed floors developed an average of 53.6% of composite action, while the glued floors developed an average of 74.8% composite action. The found that for floor systems that were completely identical except for the type of connection used, glued floors had a 15% reduction in static deflection over the nailed only floors.

The authors performed extensive testing on what effect joist variability (MOE) has on static floor performance (total stiffness of the whole floor system). They found that the variation of average floor deflection was 1/4th that of joist MOE variation. They also found that the type of floor system did not strongly influence this conclusion. The floors at Colorado State University (CSU), mentioned previously, were modeled as complete floors while the floors at the Forest Products Laboratory (FPL), where McCutcheon conducted his research, were modeled as tee-beams. The results of this simplified tee-beam method compared well with those of CSU. Therefore it was concluded that the tee-beam method is well suited to the distributed loading and stiffness criterion for their study. The results of their research indicated that a large amount of composite action exists in wood floors and should be used by the designer instead of designing on the basis of joists acting alone.

The tee-beam method was developed by McCutcheon [1977] at the FPL. The objective of his study was develop a procedure to compute the static deflection of wood floors. McCutcheon used only one layer of sheathing (this is current construction practice) and he assumed the deflections in the transverse direction are constant. Finally, he assumed a single-span with simple supports and most importantly that the effective width of the flange (plywood) was equal to the center-to-center spacing of the joists.

The author's research entailed using this tee-beam model to derive load/slip values for nailed connections, these are the k values discussed previously. From this information he developed with deflection calculations that take into account the added stiffness of the floor obtained by the semi-composite action of the connection of joist and sheathing. For 22 of the 29 computed deflection calculations that the author performed, the predicted values were within 5% of the experimentally measured values. However, his method does not account for two-way action and his equation requires empirical modification to account for the flexible gaps in the sheathing.

Ellingwood and Tallin [1984] proposed tentative serviceability criteria that would help predict objectionable floor vibrations. They stated that the use of a heel drop may be useful for evaluating the response of a floor to activities that are caused by impact forces. However, since a heel drop is an isolated transient vibration it is not a good simulation of vibration due to walking, and therefore not a good measure of floor acceptability since vibrations due to walking are the major annoyance in floor systems. They stated that the heel drop test is highly dependent on damping.

Their research is confined to only vibration resulting from occupant motion (walking only). The authors assumed that floors behave as one-way systems and that the fundamental mode of vibration dominates the response of the floor system. They developed equations that predict the modal response to a single foot fall. They then modified the equation that assumes the floor acts like a plate rather than a simple beam. A computer program was written to perform the calculation of their very complex equation.

"Resonance is the most important factor affecting vibration from rhythmic activities: hence, natural frequency is the most important structural design parameter" Allen [1990]. Allen states that the current basis of determining frequencies is based on beam flexure. However, he states that shear deformations and flexibility of supports play a big part in increasing floor flexibility and therefore reducing natural frequency of the floor.

Murray [1975, 1981, 1985, 1991] over the years has developed a straight forward and easy to use analytical procedure that determines the dynamic acceptability of steel beam (or joist) and concrete slab floor systems supporting office or residential environments. This procedure is dependent on a 'human response scale' that was derived from data obtained from tests of approximately 100 in-situ floor systems. This scale is dependent on three variables: (1) first natural frequency, (2) either maximum amplitude, velocity, for a reference excitation, and (3) damping. Murray stated that the most important variable for residential and office floor systems is damping. This is because the troublesome problem associated with vibration is due to transient motion, (short duration impact, decaying with time), i.e. walking.

The vast majority of the floor systems tested were in the range of 5 Hz to 8 Hz. Murray stated that for floor systems exhibiting first natural frequencies greater than 10 Hz, the use of his human response scale criterion is not recommended.

Murray proposed the following inequality for determining if the vibration of a floor will be a problem.

$$D > 35A_o f + 2.5 \qquad (1.18)$$

where D = damping in percent of critical,

A_o = maximum initial amplitude of the floor system due to a heel-drop excitation, in., and

f = first natural frequency of the floor system, Hz.

The excitation of the floor is performed by a 190 lb. man standing on his toes and then suddenly dropping on his heels (commonly referred to as a 'heel-drop'). This heel drop represents a linear decreasing ramp function of 600 lb. in magnitude decaying over a duration of 50 milliseconds. According to Murray, the use of this criterion demands that the designer pay special attention to the type of building and its use when estimating the percent critical damping that the floor will exhibit, because damping is highly dependent on building features such as hung ceilings, ductwork, thickness of floor slab, etc. The initial amplitude of the floor system due to a heel drop impact, A_o, is estimated by initial amplitude of a single tee-beam and dividing it by the number of effective tee-beams.

Murray [1991] pointed out that the designer should make sure the frequency of the floor system is at least greater than 3 Hz (this is to avoid resonance due to walking). Also, many internal organs of humans have natural frequencies in the range of 5 Hz to 8 Hz, and consequently many of the problem floors Murray investigated were found to have first natural frequencies of 5 Hz to 8 Hz.

To determine natural frequency of the floor system, Murray analytically modeled the floor system as a tee-beam. He assumed fully composite action between beam (or joist) and concrete slab. Calculation of natural frequency is as follows:

$$f = k\sqrt{\frac{gEI_t}{WL^3}} \quad \text{(Hz)} \qquad (1.19)$$

where

k = 1.57, $\left(\frac{\pi}{2}\right)$ for simply supported beams with uniformly distributed loading,

g = acceleration of gravity, in./sec./sec.,

E = modulus of elasticity, psi,

I_t = transformed moment of inertia of the tee-beam, in.^{4},

W = total weight supported by the tee-beam, lb., and

L = tee-beam span, in.

If the beams rest on girders, then the system frequency is used in Inequality 1.18. The system frequency can be estimated from

$$\frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} \tag{1.20}$$

where f_s = system frequency, Hz,

f_b = beam or joist frequency, Hz, and

f_g = girder frequency, Hz.

One can then draw conclusions from Murray's research and simple design criterion. One can see how frequency, amplitude, and damping all interrelate. These three factors, and how the resulting response they produce, determine if the vibration of a floor will be of concern to the inhabitants. Murray's procedure is widely used by engineers in the steel design community and there has never been reported a floor system that has satisfied the criterion and yet judged to be unacceptable by the occupants.

1.3 Scope of Work

This thesis and included research is part of research project currently sponsored by the **United States Department of Agriculture National Research Initiative Competitive Grants Program** in the area of *Improved Utilization of Wood and Wood Fiber*. The goal of this project is to improve the dynamic performance, that is primarily a serviceability condition, of wood floors in buildings. The writer's research encompasses part of the goal of this project. This project also supplements a second project that is concurrently being conducted at VPI & SU.

One goal of this project is to apply Murray's vibrational criterion to wood structures. Murray's design criterion is the most widely used criterion in the United States by design engineers for steel-concrete floor systems. His criteria has never produced erroneous results and it encompasses all the required parameters that affect human perception of floor vibration. Therefore, if this criterion can be modified for wood floors it will be more readily accepted by the design community. This criterion is based on four parameters: fundamental frequency, initial amplitude, damping, and a level of human perceptibility.

This writer's research is concerned with the first of these four parameters: **fundamental frequency**. It is the writer's goal to, with simple mathematics, be able to predict fundamental frequencies of actual wood joist floors. The end result of this goal will allow design engineers the ability to accurately predict the frequency of a floor system while still in the design stage with only information available at that stage of the design. This process is currently done with steel-concrete floors and is quite accurate. However, many of the initial parameters and assumptions in steel-concrete systems, which make the determination of frequencies quite easy, may not be true for wood structures.

Therefore it is the purpose of the writer's work to be able to predict frequencies of wood floor systems, accurately and simply. At the same time one must not lose sight of the purpose of these results. These results will be only part of a design criterion. It is therefore advantages for the writer's results to be as accurate as possible. However, on the other hand, this design criterion will be rated against a human perceptibility criterion. This 'human perceptibility criterion' will by definition have a great variance. Because of this human criterion, the ability to predict frequencies, to great accuracy, will not be of significant scientific importance when the overall scope of this research is kept in mind. Also, since the ultimate goal of this project is to produce a complete design criterion, there lies a great need to keep all the individual parameters, that will contribute to the overall design criterion, as simple as possible.

CHAPTER II

EXPERIMENTAL INVESTIGATION

2.1 Overview

Thirteen full size floors were constructed and tested for vibrational characteristics. The floors were 16 ft. x 16 ft. in plan. Each floor contained nine joists, with 23/32 in. tongue-in-groove plywood sheathing used for flooring. The sheathing was nailed and glued (visco-elastic) to the joists. Typical construction methods were used.

In addition, ten tee-beam specimens, consisting of two joists and sheathing were also tested. Ten of the tee-beams were cut from the full size floors; two were constructed from new materials.

2.2 Description of Specimens

Three types of floor joists were used: solid-sawn, parallel chord floor trusses, and composite I-joists. Figure 2.1 shows cross-sections of the floor joists. The ends of all joists were connected to wood headers by means of joists hangers, except the parallel chord trusses. Since these joists were bottom chord bearing, hangers were not needed.

The solid-sawn joists were nominal 2 in. x 12 in. No. 2 and better KD15 Southern Pine. The actual cross-section dimensions of these joists were 1.5 in. x 11.25 in.

The top and bottom chords in the parallel chord trusses were nominal 4 in. x 2 in. No. 2 Southern Pine, while the webs were nominal 4 in. x 2 in. No. 3 Southern Pine. The trusses were 12 in. deep.

Three sizes of composite I-joists were used. Each had laminated veneer lumber (LVL) flanges and oriented strand board (OSB) webs. Actual joist depths were 9.5 in.

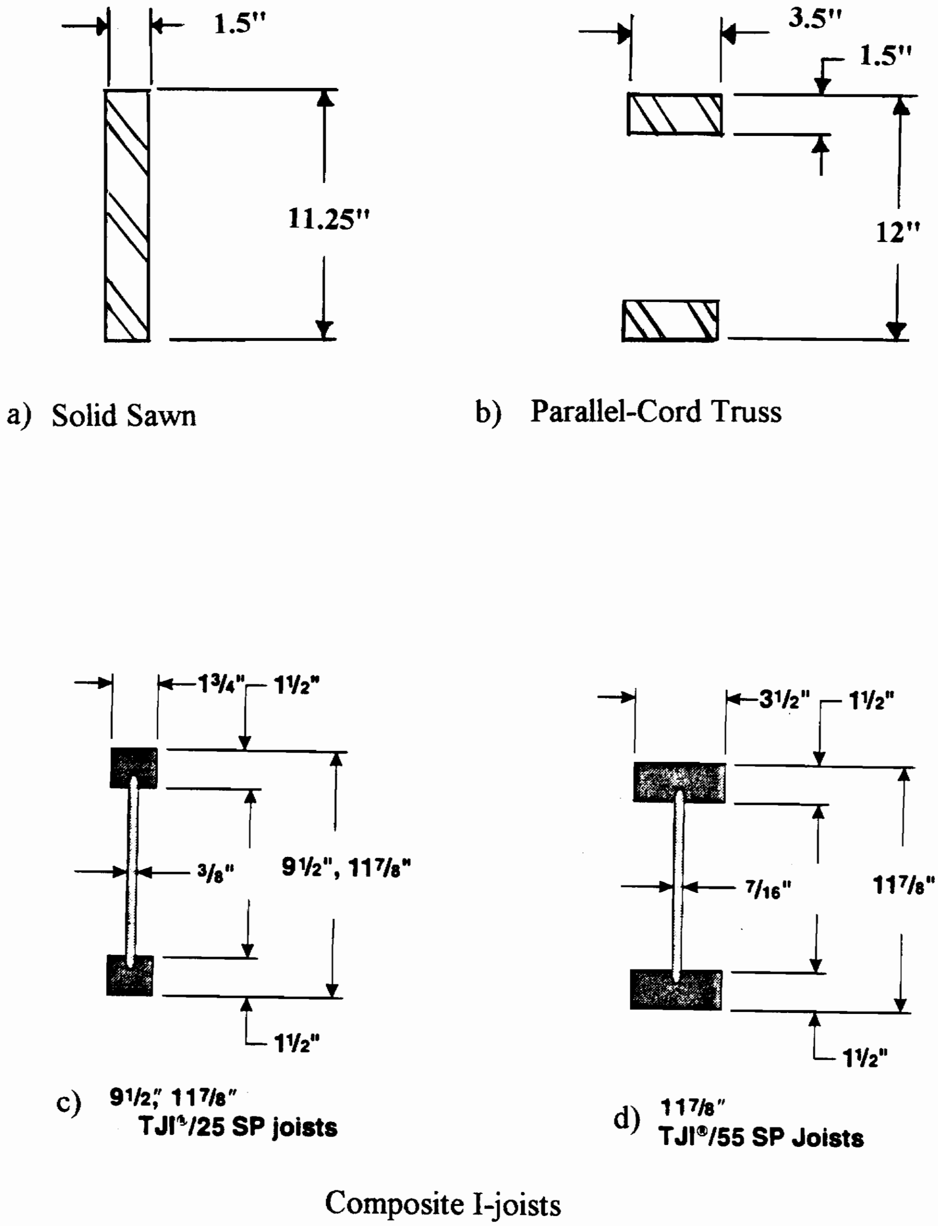


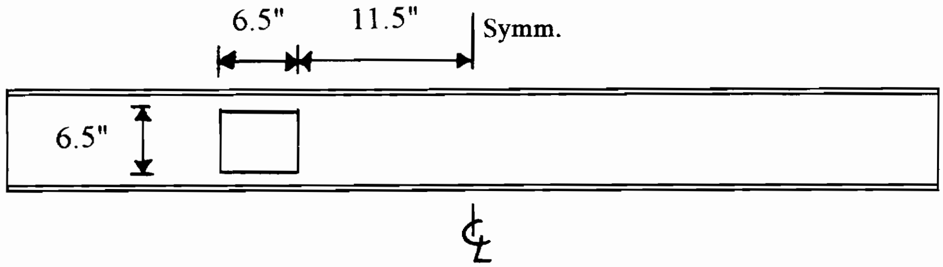
FIGURE 2.1 Joist Dimensions

and 11.875 in. Chord sizes were 1.75 in. x 1.5 in. and 3.5 in. x 1.5 in. The composite I-joist floors are hereafter referred to as 9.5 in. deep, nominal 12 in. deep small flange (1.75 in. x 1.5 in.), and nominal 12 in. deep large flange (3.5 in. x 1.5 in.) composite I-joists. In a sister research project, many test scenarios were performed using the full 16 ft. x 16 ft. floors, such as, adding bridging, blocking, strong backs, post tensioning, and cutting holes in the joists to simulate ductwork. After the full floors were tested, two double tee-beam sections were cut out of four of the floors for use in this study. As a consequence, all composite I-joists when tested as tee-beams, had square holes cut in their webs in two locations. The size and location of the holes were as specified by TRUS JOIST MacMILLAN, the joist manufacturer. The size and location of the holes is shown in Figure 2.2.

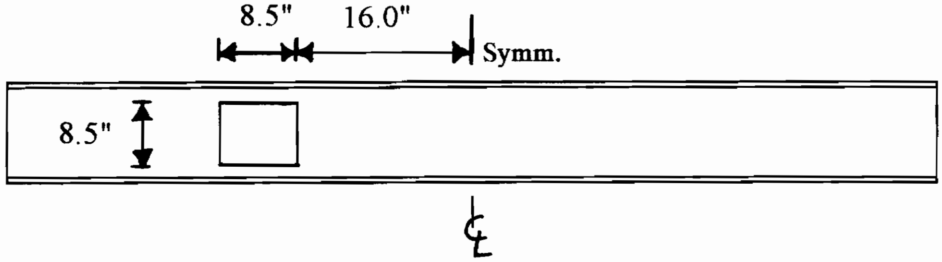
2.3 Instrumentation

2.3.1 Tee-beams

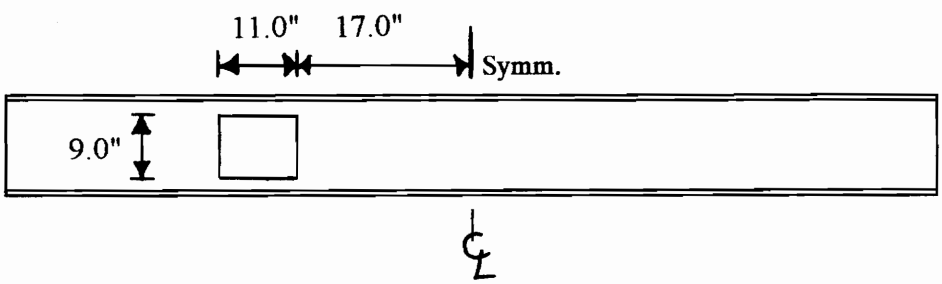
The tee-beam floors were tested on a steel frame that consisted of two very stiff steel test frame supports. The testing equipment consisted of a liquid damped seismic accelerometer connected to an amplifier box which in turn was connected to an analog-to-digital converter which was linked to a microcomputer. The equipment recorded 1024 data points over ten seconds. These acceleration traces were analyzed using a commercial fast fourier transform program. From these results a power spectrum graph was produced showing relative unit power verses frequency (Hz). Appendix A contains representative samples of the fundamental frequencies of the tee-beam floors that were obtained from these power spectra. Typical power spectra can be found in Appendix B. All the power spectra in Appendix B were obtained by measuring the acceleration of the center of the floors produced at the quarter point of the span.



a) 9.5 in. I-joist



b) Nominal 12 in. Small Flange I-joist



c) Nominal 12 in. Large Flange I-joist

FIGURE 2.2 Location of Holes in I-joists

2.3.2 Full Floors

The full floors were tested on a steel test frame consisting of A36 steel channels welded flange tip-to-flange tip to form a steel tube 8 in. wide, 10 in. deep, and 16 ft. long. Four of these tubes were then connected forming a 16 ft. x 16 ft. square frame. The frame was supported approximately 3 ft. above the laboratory floor by steel columns, bolted to the underside at each corner of the test frame. The full test frame was then further braced using steel cable counters and nominal 4 in. x 4 in. lumber. Wooden sill plates were bolted to the test frame. The wooden headers of the floors were toe-nailed into the wooden sill plates to simulate simple-simple supports.

The motion of the floors was measured using four piezoelectric accelerometers attached to the bottom of the joists. Only data from the accelerometer located at the center of the floor was used in this study. When the floor was excited, the output from the accelerometer was recorded at a rate of 1000 data points a second. Typically, data was recorded for three seconds for a total of 3000 data points. The data was recorded and then exported into a commercial software program called Dadisp [1991]. Dadisp is a program that can mathematically manipulate numbers, such a performing a fast fourier transform on acceleration traces.

2.3.3 Imposed Loading

The imposed loading used for this research consisted of canvas bags filled with steel pellets. Each bag weighed 30 lb. Tee-beam floors were tested with 0, 5, 10, and 15 psf of imposed loading. Full floor tests were conducted with 0, 20, and 40 psf imposed loading.

2.4 Testing Procedures and Results

2.4.1 Tee-beams

The testing equipment used for the tee-beam floors was originally developed to test steel-concrete floors. Two testing modifications were made because of this fact. The first was that the accelerometer was attached to a 10 lb. steel plate, so it would not "walk" across the floor. The second was that a standard heel drop could not be used, thus not simulating the standard 600 lb. ramp function used in the Murray criteria (see Section 1.2.3) and shown in Figure 2.3. Only a 1/3rd heel drop could be used so as not to exceed the limits of the testing equipment. A 1/3rd heel drop was performed by lifting the heels 1/3rd as high as one would for a full heel drop.

To conduct a test, the writer, a 185 lb. male, would stand on the tee-beam and raise up approximately 1/3rd as high as a full heel drop. The recording equipment would be activated and approximately one second later the writer's heels would impact the floor. During the ten seconds from when the equipment was activated, the minicomputer would record 1024 data points, referred to as the acceleration trace.

The typical process of testing a tee-beam floor is as follows: the accelerometer and the excitation force were always applied along the centerline of the floor parallel, and centered between the joists. The accelerometer would be placed at a quarter point on the floor and then the writer would perform two heel drops next to the accelerometer, two heel drops at the center of the floor, and then two heel drops at the opposite quarter point. The accelerometer would then be moved to the center of the floor (again along the center line), and then the same heel drop scenario would be performed: two heel drops at one end of the floor at a quarter point, two heel drops at the center of the floor, and two heel drops at the opposite quarter point. The accelerometer would then be moved to the opposite quarter point and the succession of six heel drops would take place again.

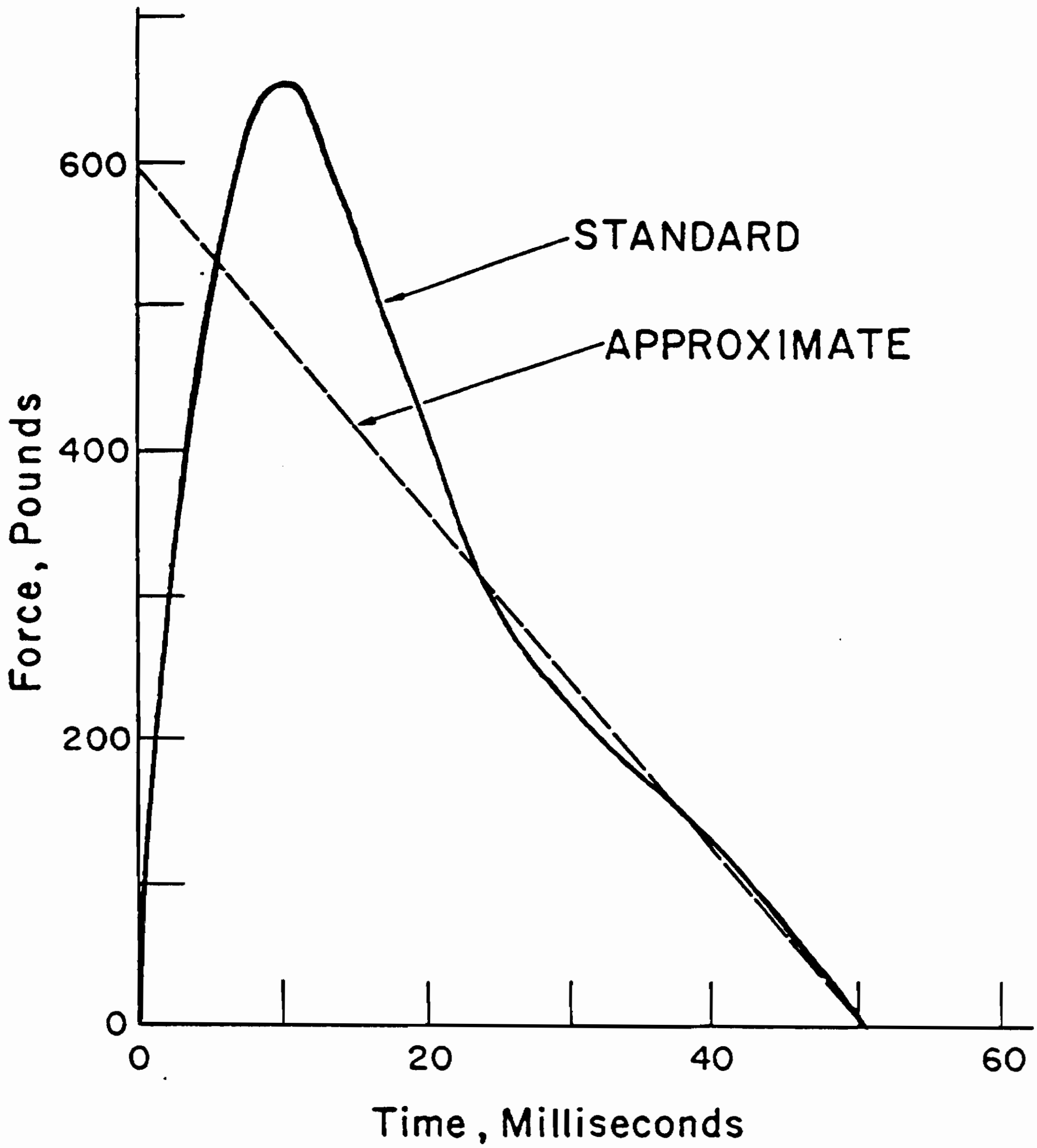


FIGURE 2.3 Heel Drop Impact

The complete scenario was then repeated with 5, 10, and 15 psf imposed loading. The reason duplicate measurements were taken was to determine how repeatable a 1/3rd heel drop was. Since the only information wanted from these tests was frequency, a 1/3rd heel drop excitation was found to be a satisfactory method of excitation.

After careful study of the power spectra produced from the results of the tee-beams tested, the determination was made that the first significantly large power spike was the fundamental frequency of the tee-beams. All the smaller spikes before this one were attributed to noise and/or a torsional mode of vibration of the tee-beams. Once dead load was added to the tee-beams the power spectra became much more 'clear' and the torsional mode usually combined with the fundamental mode and the relative unit power sometimes grew ten times as large. By averaging the frequencies obtained by numerous identical testing of the tee-beams, the fundamental frequency under each case of imposed loading was determined.

2.4.2 Full Floors

The full floors were excited by a mechanical impactor (drop weight) that simulates a human heel drop. This produces a forcing function similar to that shown in Figure 2.3. The floors were also tested by suddenly releasing a 700 lb. weight supported from the bottom of the center joist which produced a free vibration of the floors. The floors were tested under 0, 20, and 40 psf imposed loading.

Comparisons were made between the frequency responses of the floors when excited by the mechanical impactor and the 700 lb. weight. It was found that the floor response differed depending on the excitation force used. The reason for the differences was due to the weight of the mechanical impactor, approximately 146 lb., that increased the imposed load of the floor in addition to the uniform loads applied. It would be expected that the frequencies obtained by the mechanical impactor would be lower than those obtained from the free vibration tests. One would also expect that as more imposed load was added to the floor (20 psf and 40 psf) the frequencies, produced by the two methods of excitation, would become much closer since the additional 146 lb. weight of the mechanical impactor would be a much smaller percentage of the total weight of the system. Both of these expectations were found to be true.

It was found that the steel test frame had significant impact on the vibrational performance of the full floors. This was verified by placing one of the full floors on the concrete lab floor and retesting it. An attempt to find the vibrational characteristics of the frame was made, but the results were inconclusive.

Several tee-beam floors with known frequency values were tested on the full floor test frame. With 0 psf imposed loading, the frequencies obtained on the steel frame were significantly less than the actual frequencies of the tee-beams. However, as 5 psf and 10 psf imposed load was applied to the tee-beams, the resulting frequencies for the specimens on the steel frame became much closer to the frequencies of the tee-beams, comparisons are shown in Figures B.1 (e) through (j), and B.2 (a) through (f). The conclusion of this scenario proved that the test frame had significantly influenced the frequencies of the full floors when no imposed loading was used, but the effect of the frame diminished as more imposed load was placed on the frame. Due to the complex vibrational nature of the influence of the frame on the full floors, the frequency of the full floors themselves could not be separated out of the data obtained when no imposed load was placed on the full floors. When 20 psf and 40 psf imposed loading was placed on the full floors, the measured frequencies appeared to be those of the full floors alone, and therefore correct and usable.

CHAPTER III

FREQUENCY PREDICTIONS AND COMPARISONS

3.1 Basis of Predictions

The equation used in this research to predict fundamental frequency of a tee-beam is

$$f = k\sqrt{\frac{gEI_e}{WL^3}} \quad \text{(Hz)} \tag{3.1}$$

where f = frequency (Hz),

$k = 1.57, \left(\frac{\pi}{2}\right)$ for simply supported beams,

g = acceleration of gravity, in./sec./sec.,

E = modulus of elasticity, psi,

I_e = effective moment of inertia of the tee-beam, in.^{4},

W = total weight supported by the tee-beam, lb., and

L = tee-beam span, in.,.

Equation 3.1 requires the effective stiffness of the tee-beam, EI_e, which in turn requires the effective width of the sheathing. For vibration calculations of steel-concrete floor systems, an effective slab width equal to the beam spacing is used to calculate the effective moment of inertia and is referred to as the transformed moment of inertia. For wood joist floor systems it has long been known that the effective width of the sheathing is considerably less than the joist spacing. The effective width of the sheathing was determined as described in the Section 3.3.

3.2 Frequency Predictions using Experimentally Determined EI_e

To experimentally determine the effective moment of inertia of each tee-beam joist, EI_e, a dial gauge was placed under both joists at midspan of each tee-beam. The tee-beam was then uniformly loaded along its length. The resulting deflection was used in

$$EI_e = \frac{5wl^4}{384\Delta} \quad (\text{kip-in.}^2) \tag{3.2}$$

where EI_e = effective floor stiffness, kip-in.^{2},

w = uniformly distributed load per unit of length, kip/in.,

l = span length, in., and

Δ = measured deflection, in.

to determine the effective floor stiffness. This value was then used in Equation 3.1 to predict the floor frequency. Predicted and measured frequencies of the tee-beams are shown in Table 3.1, along with the ratio of the predicted-to-measured values. The mean of the ratios is 0.955 and the standard deviation is 0.048.

3.3 Effective Width Determination

To determine the effective sheathing width, the moment of inertia of each joist type was first determined either by direct calculation (solid-sawn), from standard industry practice (parallel chord trusses where the moment of inertia is taken as 75% of the moment of inertia of the top and bottom chords), or from direct calculation using measured dimensions (composite I-joists). Next, a modulus of elasticity was determined from published data for the particular joist type. Tabulated values of the modulus of elasticity, E, were used for the solid-sawn and parallel chord trusses and for the sheathing. Values of E for the solid-sawn lumber and parallel chord trusses were obtained from the National Design Specification for Wood Construction Supplement [1991] *Design Values*

TABLE 3.1 Predicted Tee-beam Frequencies Using Measured Ele Data

Joist Type	Imposed Load (psf)	Average Measured Ele (kip - in.^2)	Predicted Frequency Based on Measured Ele (Hz)	Measured Frequency (Hz)	Predicted / Measured
Solid-Sawn Number One	0	385300	19.8	20.4	0.971
	5	"	13.2	14.4	0.917
	10	"	10.8	10.6	1.019
	15	"	9.2	9.9	0.929
Solid-Sawn Number Two	0	468800	22.3	20.7	1.077
	5	"	14.8	14.5	1.021
	10	"	12.1	12.4	0.976
	15	"	10.3	10.9	0.945
Parallel Cord Truss Number One	0	343100	18.8	20.5	0.917
	5	"	12.5	13.8	0.906
	10	"	10.2	10.9	0.936
	15	"	8.7	9.6	0.906
Parallel Cord Truss Number Two	0	411600	20.5	21.2	0.967
	5	"	13.7	14.4	0.951
	10	"	11.1	11.3	0.982
	15	"	9.5	9.9	0.960
Composite I-joists with holes					
9.5 in. I-joist Number One	0	208500	16.5	16.8	0.982
	5	"	10.2	10.6	0.962
	10	"	8.2	8.4	0.976
	15	"	6.9	7.2	0.958
9.5 in. I-joist Number Two	0	209200	16.5	16.9	0.976
	5	"	10.2	10.5	0.971
	10	"	8.2	8.6	0.953
	15	"	6.9	7.3	0.945
Nominal 12 in. Small Flange I-joist Number One	0	347500	20.8	21.1	0.986
	5	"	13.1	14.5	0.903
	10	"	10.5	12.1	0.868
	15	"	8.9	10.5	0.848
Nominal 12 in. Small Flange I-joist Number Two	0	359400	21.1	21.3	0.991
	5	"	13.3	14.5	0.917
	10	"	10.7	12.2	0.877
	15	"	9.1	10.6	0.858

TABLE 3.1 Predicted Tee-beam Frequencies Using Measured Ele Data, Continued

Joist Type	Imposed Load (psf)	Average Measured Ele (kip - in.^2)	Predicted Frequency Based on Measured Ele (Hz)	Measured Frequency (Hz)	Predicted / Measured
Nominal 12 in.	0	455200	21.4	21.3	1.005
Large Flange	5	"	14.3	14.4	0.993
I-joist	10	"	11.7	11.9	0.983
Number One	15	"	10	10.9	0.917
Nominal 12 in.	0	468500	21.8	21.5	1.014
Large Flange	5	"	14.5	14.5	1.000
I-joist	10	"	11.9	11.8	1.008
Number Two	15	"	10.1	11	0.918
				Mean =	**0.955**
				Standard Deviation =	**0.048**

for Wood Construction (NDS). The resulting stiffness was then calculated. The stiffnesses of the composite I-joists were obtained from a TRUS JOIST MacMILLAN publication *Specifier's Guide to the Silent Floor System* [1992] from which an effective modulus of elasticity was calculated. The axial sheathing stiffness was obtained from the American Plywood Association Technical Note N375A [1991]. This data are summarized in Table 3.2.

Next a computer program was written to calculate and plot the effective moment of inertia versus effective sheathing width for each joist type ($I_{effective}$ verses $b_{effective}$) as shown in Figure 3.1. Using the modulus of elasticity values from Table 3.2 and the experimentally determined EI_e values, the actual effective moments of inertia, I_e, were calculated. The effective sheathing widths were then determined as illustrated in Figure 3.1.

Table 3.3 is a summary of the above calculations. For the solid-sawn tee-beams, the effective widths varied between 12.7 in. and more than 24 in. (larger than the available sheathing width, 24 in.). For the parallel chord trusses, the effective width varied from less than 0 in. (less than the top chord width, 3.5 in.) to 8.6 in. For the 9.5 in. and 12 in. small flange I-joists, the effective width varied from less than 0 in. (less than the top flange width, 1.75 in.) to 4.8 in. For the nominal 12 in. large flange I-joist, the effective widths were all less than zero. Thus, it is obvious that the results in Table 3.3 are not valid. Two reasons are offered: (1) tabulated properties were used in lieu of measured properties and (2) the effect of the web holes in the composite I-joists was not considered.

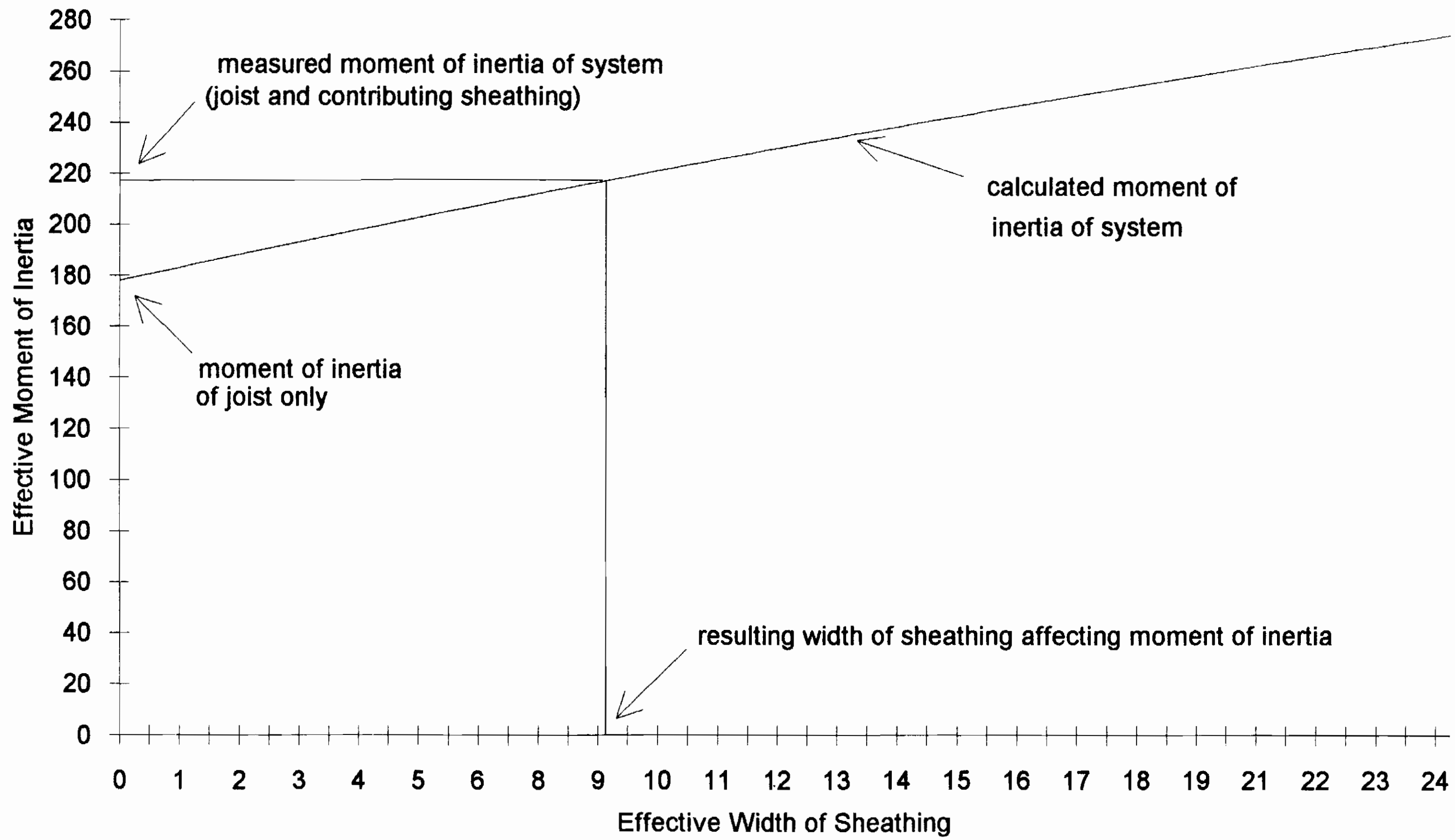


FIGURE 3.1 $I_{effective}$ (in.^{4}) versus $b_{effective}$ (in.)

TABLE 3.2 Nominal Joist Properties

Material	Moment of Inertia (in.^4)	Modulus of Elasticity (psi)	Stiffness (kip - in.^2)
Solid-Sawn	178	1,600,000	284,800
Parallel Chord Truss	219	1,600,000	350,400
9.5 in. I-joist	94	2,021,000	190,000
Nominal 12 in. Small Flange I-joist	164	2,006,000	329,000
Nominal 12 in. Large Flange I-joist	310	2,042,000	633,000
Sheathing 23/32 in.	(in.^4 / ft.) 0	(psi) 325,000	(lb. / ft.) 2,800,000

TABLE 3.3 Experimentally Determined Effective Sheathing Widths

Joist Type	Tee-Beam	Joist	Measured Ele (kip - in.^2)	E (psi)	Ie (in.^4)	b(effective) (in.)
Solid-Sawn	Number	1	390,200	1,600,000	243.9	14.1
	One	2	380,400	1,600,000	237.8	12.7
	Number	1	446,500	1,600,000	279.1	23.2
	Two	2	491,100	1,600,000	306.9	>24
Parallel Chord	Number	1	323,800	1,600,000	202.4	<0
Truss	One	2	362,300	1,600,000	226.4	1.3
	Number	1	422,700	1,600,000	264.2	8.6
	Two	2	400,500	1,600,000	250.3	5.7
Composite I-joists With Holes						
9.5 in.	Number	1	211,400	2,021,000	104.6	3.7
I-joist	One	2	205,600	2,021,000	101.7	2.7
	Number	1	210,500	2,021,000	104.2	3.5
	Two	2	207,900	2,021,000	102.9	3.01
Nominal 12 in.	Number	1	371,200	2,006,000	185.0	4.8
Small Flange	One	2	323,800	2,006,000	161.4	<0
I-joist	Number	1	352,800	2,006,000	175.9	2.7
	Two	2	365,900	2,006,000	182.4	4.2
Nominal 12 in.	Number	1	475,500	2,042,000	232.9	<0
Large Flange	One	2	434,800	2,042,000	212.9	<0
I-joist	Number	1	447,800	2,042,000	219.3	<0
	Two	2	489,200	2,042,000	239.6	<0

3.4 Design Approximation and Predicted Frequencies

In the American Concrete Institute Building Code [1989] (ACI 318-89) the effective width of the flange of a concrete tee-beam is to be taken as the minimum of the following: (1) beam spacing (2) 1/4th beam span (3) 16 times the slab thickness plus the beam flange width. For the tee-beams used in this study, the controlling width was criterion (3), that is,

$$b_{effective} = 16t_s + b_f \qquad (3.3)$$

where $b_{effective}$ = contributing width of the sheathing, in.,

$16t_s$ = 16 times the sheathing thickness, in., and

b_f = the flange width of the joist, in.

The resulting effective widths, effective moments of inertia, I_e, and effective joist stiffnesses, EI_e, for each joist type are shown in Table 3.4. The corresponding predicted frequencies for the ten tee-beams tested are shown in Table 3.5, along with the ratio of the predicted-to-measured values. The mean of the ratios is 1.059 and the standard deviation is 0.112.

3.5 Summary of Results and Comparisons

Table 3.6 presents a side by side comparison of the tee-beam floors obtained by the three different methods. All three methods compared well except for the composite I-joist floors. For these floors the frequencies obtained by stiffness calculation and accelerometer compared well, but the predicted frequencies were considerably higher. One reason for this is that the method used to obtain the predicted frequencies does not take into account the holes that were present in the composite I-joists.

Measured and predicted frequencies of the full size floors are shown in Table 3.7, along with the ratio of the predicted-to-measured values. The mean of the ratios is 1.046

and the standard deviation is 0.094. The measured frequency values of the full floors were obtained before the composite I-joists had holes cut in them. Consequently, the two frequency values compared well. Except for the nominal 12 in. large flange composite I-joist. Again, tabulated values were used in lieu of measured properties, as discussed previously in Section 3.3.

TABLE 3.4 Effective Tee-beam Properties using 16t + bf

Joist Type	Effective Width 16t + bf (in.)	Ie (in.^4)	E (Table 3.2) (psi)	EIe (kip - in.^2)
Solid-Sawn	13	239.4	1,600,000	383,040
Parallel Chord Truss	15	292.4	1,600,000	467,840
9.5 in. I-joist	13.25	127.7	2,021,000	258,082
Nominal 12 in. Small Flange I-joist	13.25	216.5	2,006,000	434,299
Nominal 12 in. Large Flange I-joist	15	372.3	2,042,000	760,237

TABLE 3.5 Predicted Tee-beam Frequencies Using 16t + bf

Joist Type	Imposed Load (psf)	Average Predicted Ele (Table 3.4) (kip - in.^2)	Predicted Frequency Based on Ele (Hz)	Measured Frequency (Hz)	Predicted / Measured
Solid-Sawn Number One	0	383040	19.8	20.4	0.971
	5	"	13.2	14.4	0.917
	10	"	10.7	10.6	1.009
	15	"	9.2	9.9	0.929
Solid-Sawn Number Two	0	383040	20.1	20.7	0.971
	5	"	13.4	14.5	0.924
	10	"	10.9	12.4	0.879
	15	"	9.3	10.9	0.853
Parallel Cord Truss Number One	0	467840	21.9	20.5	1.068
	5	"	14.6	13.8	1.058
	10	"	11.9	10.9	1.092
	15	"	10.1	9.6	1.052
Parallel Cord Truss Number Two	0	467840	21.9	21.2	1.033
	5	"	14.6	14.4	1.014
	10	"	11.9	11.3	1.053
	15	"	10.1	9.9	1.020
Composite I-joists with holes					
9.5 in. I-joist Number One	0	258082	18.2	16.8	1.083
	5	"	11.3	10.6	1.066
	10	"	9.1	8.4	1.083
	15	"	7.7	7.2	1.069
9.5 in. I-joist Number Two	0	258082	18.2	16.9	1.077
	5	"	11.3	10.5	1.076
	10	"	9.1	8.6	1.058
	15	"	7.7	7.3	1.055
Nominal 12 in. Small Flange I-joist Number One	0	434299	23.2	21.1	1.100
	5	"	14.6	14.5	1.007
	10	"	11.7	12.1	0.967
	15	"	9.9	10.5	0.943
Nominal 12 in. Small Flange I-joist Number Two	0	434299	23.2	21.3	1.089
	5	"	14.6	14.5	1.007
	10	"	11.7	12.2	0.959
	15	"	9.9	10.6	0.934

TABLE 3.5 Predicted Tee-beam Frequencies Using 16t + bf, Continued

Joist Type	Imposed Load (psf)	Average Predicted Ele (Table 3.4) (kip - in.^2)	Predicted Frequency Based on Ele (Hz)	Measured Frequency (Hz)	Predicted / Measured
Nominal 12 in.	0	760237	27.4	21.3	1.286
Large Flange	5	"	18.3	14.4	1.271
I-joist	10	"	14.9	11.9	1.252
Number One	15	"	12.8	10.9	1.174
Nominal 12 in.	0	760237	27.4	21.5	1.274
Large Flange	5	"	18.3	14.5	1.262
I-joist	10	"	14.9	11.8	1.263
Number Two	15	"	12.8	11	1.164
				Mean =	**1.059**
				Standard Deviation =	**0.112**

TABLE 3.6 Summary of Tee-beam Frequencies

Joist Type	Imposed Load (psf)	Predicted Frequency Based on Average Measured Ele (Hz)	Measured Frequency (Hz)	Predicted Frequency Based on 16t + bf (Hz)
Solid-Sawn Number One	0	19.8	20.4	19.8
	5	13.2	14.4	13.2
	10	10.8	10.6	10.7
	15	9.2	9.9	9.2
Solid-Sawn Number Two	0	22.3	20.7	20.1
	5	14.8	14.5	13.4
	10	12.1	12.4	10.9
	15	10.3	10.9	9.3
Parallel Cord Truss Number One	0	18.8	20.5	21.9
	5	12.5	13.8	14.6
	10	10.2	10.9	11.9
	15	8.7	9.6	10.1
Parallel Cord Truss Number Two	0	20.5	21.2	21.9
	5	13.7	14.4	14.6
	10	11.1	11.3	11.9
	15	9.5	9.9	10.1
Composite I-joists with holes				
9.5 in. I-joist Number One	0	16.5	16.8	18.2
	5	10.2	10.6	11.1
	10	8.2	8.4	9.1
	15	6.9	7.2	7.7
9.5 in. I-joist Number Two	0	16.5	16.9	18.2
	5	10.2	10.5	11.1
	10	8.2	8.6	9.1
	15	6.9	7.3	7.7
Nominal 12 in. Small Flange I-joist Number One	0	20.8	21.1	23.2
	5	13.1	14.5	14.6
	10	10.5	12.1	11.7
	15	8.9	10.5	9.9
Nominal 12 in. Small Flange I-joist Number Two	0	21.1	21.3	23.2
	5	13.3	14.5	14.6
	10	10.7	12.2	11.7
	15	9.1	10.6	9.9

TABLE 3.6 Summary of Tee-beam Frequencies, Continued

Joist Type	Imposed Load (psf)	Predicted Frequency Based on Average Measured Ele (Hz)	Measured Frequency (Hz)	Predicted Frequency Based on 16t + bf (Hz)
Nominal 12 in.	0	21.4	21.3	27.4
Large Flange	5	14.3	14.4	18.3
I-joist	10	11.7	11.9	14.9
Number One	15	10	10.9	12.8
Nominal 12 in.	0	21.8	21.5	27.4
Large Flange	5	14.5	14.5	18.3
I-joist	10	11.9	11.8	14.9
Number Two	15	10.1	11	12.8

TABLE 3.7 Full Floor Frequencies

Joist Type	Imposed Load (psf)	Measured Frequency (Hz)	Predicted Frequency Based on 16t + bf (Hz)	Predicted / Measured
Solid-Sawn	20	8.3	8.2	0.988
	40	6.7	6	0.896
Parallel Cord Truss	20	8.7	9	1.034
	40	6.3	6.7	1.063
Composite I-joists				
9.5 in. I-joist	20	7	6.8	0.971
	40	5.2	5	0.962
Nominal 12 in. Small Flange I-joist	20	8.3	8.8	1.060
	40	6	6.5	1.083
Nominal 12 in. Large Flange I-joist	20	9.5	11.4	1.200
	40	7	8.4	1.200
			Mean =	**1.046**
			Standard Deviation =	**0.094**

CHAPTER IV

RECOMMENDATIONS AND DESIGN EXAMPLES

4.1 Recommendations.

It is recommended that an effective width of the sheathing be used when calculating the tee-beam moment of inertia for determining the fundamental frequency of wood floors. It is also recommended that the axial stiffness of the sheathing be used when determining the effective moment of inertia. The use of Equation 3.3 to predict effective width along with Equation 3.1 to predict the fundamental frequency has been shown to produce favorable results and is recommended for design use.

Further testing of both tee-beam and full floors, of different plan dimensions and beam spacings than used for this research is recommended. Testing in-situ floors and comparing the results to the design recommendations should also be undertaken. It is also recommended that it be determined if the first natural frequency is the only mode that needs to be considered in a design criterion for wood floors.

4.2 Design Examples

The following design examples illustrate the above design recommendations.

Example (1) Solid-Sawn Joists. A floor system 16 ft. in length consists of 2 in. x 12 in. joists 24 in. on center with 23/32 in. tongue-in-groove sheathing which is nailed and glued to the joists. Assume the joists are No. 2 S-Dry Southern Pine and that the floor has imposed loading of 10 psf. Calculate the first natural frequency.

Tabulated joist and sheathing values obtained from the National Design Specification for Wood Construction Supplement [1991] *Design Values for Wood Construction* and the American Plywood Association Technical Note N375A [1991], respectively:

No.2 Southern Pine

$E_{joist} = 1.6 \text{ x } 10^6$ psi

$I_{joist} = 178$ in.^{4}

$W_{joist} = 64.3$ lb. (ρ =34.3 pcf)

32 / 32 *Sheathing*

$E_{sheathing} = 3.25 \text{ x } 10^5$ psi (axial)

$W_{sheathing} = 2.1$ psf (16 ft. x 2 ft.)

= 67.2 lb.

$W_{imposed} = 10$ psf (16 ft.x 2 ft.= 32 sq.ft.)

= 320 lb.

Determine $b_{effective}$ of the sheathing:

$$b_{effective} = 16t \text{ x } b_f$$

with

$t = {}^{23}\!/_{32}$ in. = 0.71875 in.

$b_f = 1.5$ in.

$$b_{effective} = 16 \text{ x } 0.71875 + 1.5$$

$$= 13.0 \text{ in.}$$

Determine the effective moment of inertia of tee-beam:

$$n = \frac{E_{joist}}{E_{sheathing}} = \frac{1.6 \times 10^6}{3.25 \times 10^5} = 4.92$$

$$b_{transformed} = \frac{b_{effective}}{n} = \frac{13}{4.92} = 2.64 \text{ in.}$$

$$\bar{y} = \frac{(2.64 \times {}^{23}\!/_{32})\,(11.25 + \frac{{}^{23}\!/_{32}}{2}) + (11.25 \times 1.5)\,(\frac{11.25}{2})}{(2.46 \times {}^{23}\!/_{32}) + (11.25 \times 1.5)}$$

$$= 6.27 \text{ in.}$$

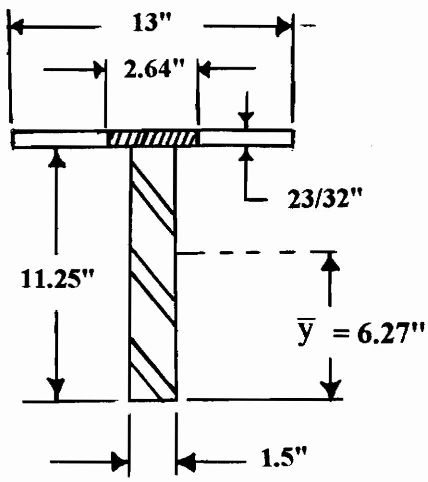


Figure 4.1 Design Example

$$I_{effective} = \frac{2.64 \times ({}^{23}\!/_{32})^3}{12} + 2.64 \times {}^{23}\!/_{32} \times (11.25 + \frac{{}^{23}\!/_{32}}{2} - 6.27)^2 +$$

$$178 + 11.25 \times 1.5 \times (6.27 - \frac{11.25}{2})^2$$

$$= 239 \text{ in.}^4$$

Use Equation 3.1 to predict frequency:

$$f = 1.57\sqrt{\frac{gEI_e}{WL^3}}$$

with

$g = 386\ ^{in}/_{sec.^2}$

$E = 1.6 \times 10^6$ psi

$I_e = 239$ in.^{4}

$W = W_{joist} + W_{sheathing} + W_{imposed\ loading}$

$= 64.3$ lb. $+ 67.2$ lb. $+ 320$ lb.

$= 452$ lb.

$L = 16$ ft. $= 192$ in.

$$f = 1.57\sqrt{\frac{386 \times (1.6 \times 10^6) \times 239}{452 \times (192)^3}}$$

$= 10.7$ Hz

Example (2) Parallel Chord Truss Joists. Repeat Example (1) using a 12 in. deep parallel chord truss with 3.5 in. wide by 1.5 in. deep No. 2 Southern Pine chords.

Determine the joist moment of inertia:

$$\begin{aligned} I_{joist} &= 0.75\ I_{chords} \\ &= 0.75\left[2 \times 3.5 \times {}^{1.5^3}/_{12} + 2 \times 3.5 \times 1.5\ ({}^{12.0}/_{2} - {}^{1.5}/_{2})^2\right] \\ &= 219 \text{ in.}^4 \end{aligned}$$

Determine the effective moment of inertia of the tee-beam:

$$n = \frac{E_{joist}}{E_{sheathing}} = \frac{1.6 \text{ x } 10^6}{3.25 \text{ x } 10^5} = 4.92$$

$$b_{transformed} = \frac{b_{effective}}{n} = \frac{15}{4.92} = 3.05 \text{ in.}$$

$$\bar{y} = \frac{(3.05 \text{ x } {}^{23}\!/_{32})\ (12.0 + \frac{{}^{23}\!/_{32}}{2}) + (2 \text{ x } 3.5 \text{ x } 1.5)\ (12.0/2)}{(3.05 \text{ x } {}^{23}\!/_{32}) + (2 \text{ x } 3.5 \text{ x } 1.5)}$$

$$= 7.10 \text{ in.}$$

$$I_{effective} = \frac{3.05 \text{ x } ({}^{23}\!/_{32})^3}{12} + 3.05 \text{ x } {}^{23}\!/_{32} \text{ x } (12.0 + \frac{{}^{23}\!/_{32}}{2} - 7.10)^2 +$$
$$219 + 2 \text{ x } 3.5 \text{ x } 1.5 \text{ x } (7.10 - \frac{12.0}{2})^2$$

$$= 292 \text{ in.}^4$$

Use Equation 3.1 to predict frequency:

$$f = 1.57\sqrt{\frac{gEI_e}{WL^3}}$$

with

$g = 386\ ^{in}/_{sec.^2}$

$E = 1.6 \times 10^6$ psi

$I_e = 239$ in.^{4}

$W = W_{joist} + W_{sheathing} + W_{imposed\ loading}$

$= 64$ lb.$+ 67.2$ lb.$+ 320$ lb.

$= 451$ lb.

$L = 16$ ft.$= 192$ in.

$$f = 1.57\sqrt{\frac{386 \times (1.6 \times 10^6) \times 292}{451 \times (192)^3}}$$

$= 11.8$ Hz

Example (3) Composite I-joist. Repeat Example (1) using a 9.5 in deep composite I-joist which has a 3/8 in. thick web and flanges 1.75 in. wide and 1.5 in. deep. Web consists of OSB while the flanges are LVL.

Determine the joist moment of inertia:

$$I_{joist} = 2 \times 1.75 \times {}^{1.5^3}/_{12} + 2 \times 1.75 \times 1.5\ ({}^{9.5}/_{2} - {}^{1.5}/_{2})^2 + {}^{3}/_{8} \times {}^{(9.5 - 2 \times 1.5)^3}/_{12}$$

$= 94$ in.^{4}

Determine the effective moment of inertia of the tee-beam:

$$n = \frac{E_{joist}}{E_{sheathing}} = \frac{2.0 \text{ x } 10^6}{3.25 \text{ x } 10^5} = 6.15$$

$$b_{transformed} = \frac{b_{effective}}{n} = \frac{13.25}{6.15} = 2.15 \text{ in.}$$

$$\bar{y} = \frac{(2.15 \text{ x } {}^{23}\!/_{32})\ (9.5 + \frac{{}^{23}\!/_{32}}{2}) + (2 \text{ x } 1.75 \text{ x } 1.5)\ \left(9.5/2\right) + (9.5 - 2 \text{ x } 1.5)\ ({}^{3}\!/_{8})\ (9.5/2)}{(2.15 \text{ x } {}^{23}\!/_{32}) + (2 \text{ x } 1.75 \text{ x } 1.5) + (9.5 - 2 \text{ x } 1.5)\ ({}^{3}\!/_{8})}$$

$$= 5.61 \text{ in.}$$

$$I_{effective} = \frac{2.15 \text{ x } ({}^{23}\!/_{32})^3}{12} + 2.15 \text{ x } {}^{23}\!/_{32} \text{ x } (9.5 + \frac{{}^{23}\!/_{32}}{2} - 5.61)^2 +$$

$$94 + 2 \text{ x } 1.75 \text{ x } 1.5\ (9.5 - 2 \text{ x } 1.5)\ ({}^{3}\!/_{8})\ (5.61 - \frac{9.5}{2})^2$$

$$= 131 \text{ in.}^4$$

Use Equation 3.1 to predict frequency:

$$f = 1.57\sqrt{\frac{gEI_e}{WL^3}}$$

with

$g = 386\ {}^{in}\!/_{sec.^2}$

$E = 2.0 \text{ x } 10^6 \text{ psi}$

$I_e = 131 \text{ in.}^4$

$W = W_{joist} + W_{sheathing} + W_{imposed\ loading}$

$= 37 \text{ lb.} + 67.2 \text{ lb.} + 320 \text{ lb.}$

$= 424 \text{ lb.}$

$L = 16 \text{ ft.} = 192 \text{ in.}$

$$f = 1.57\sqrt{\frac{386 \text{ x } (2.0 \text{ x } 10^6) \text{ x } 131}{424 \text{ x } (192)^3}}$$

$= 9.1 \text{ Hz}$

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APPENDIX A

Measured Tee-beam Fundamental Frequencies

TABLE A.1 Tee-beam Fundamental Frequencies, 2 in. x 12 in. Number One, No Holes

Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
North Quarter	North Quarter	0	20.3	n
North Quarter	North Quarter	0	17.6	y
Center of Floor	North Quarter	0	error	error
Center of Floor	North Quarter	0	17.6	n
South Quarter	North Quarter	0	17.5	y
South Quarter	North Quarter	0	17.6	y
North Quarter	Center of Floor	0	20.2	n
North Quarter	Center of Floor	0	17.6	n
Center of Floor	Center of Floor	0	17.5	n
Center of Floor	Center of Floor	0	20.6	n
South Quarter	Center of Floor	0	20.4	y
South Quarter	Center of Floor	0	17.5	y
North Quarter	North Quarter	5	14.3	y
North Quarter	North Quarter	5	14	n
Center of Floor	North Quarter	5	11.6	n
Center of Floor	North Quarter	5	14	n
South Quarter	North Quarter	5	14.5	y
South Quarter	North Quarter	5	14.3	n
North Quarter	Center of Floor	5	14.3	n
North Quarter	Center of Floor	5	11.81	y
Center of Floor	Center of Floor	5	11.31	n
Center of Floor	Center of Floor	5	11.71	n
South Quarter	Center of Floor	5	14.7	n
South Quarter	Center of Floor	5	14	n
North Quarter	North Quarter	10	11.6	n
North Quarter	North Quarter	10	11.5	n
Center of Floor	North Quarter	10	11.4	n
Center of Floor	North Quarter	10	11.4	n
South Quarter	North Quarter	10	11.5	n
South Quarter	North Quarter	10	11.5	n
North Quarter	Center of Floor	10	9.9	y
North Quarter	Center of Floor	10	11.5	y
Center of Floor	Center of Floor	10	10.4	y
Center of Floor	Center of Floor	10	10.4	y
South Quarter	Center of Floor	10	10	y
South Quarter	Center of Floor	10	10.5	n
North Quarter	North Quarter	15	10.3	y
North Quarter	North Quarter	15	error	error
Center of Floor	North Quarter	15	10	y
Center of Floor	North Quarter	15	9.9	y
South Quarter	North Quarter	15	10	n
South Quarter	North Quarter	15	10	n
North Quarter	Center of Floor	15	10	n
North Quarter	Center of Floor	15	9.9	n
Center of Floor	Center of Floor	15	9.7	y
Center of Floor	Center of Floor	15	9.7	y
South Quarter	Center of Floor	15	10	y
South Quarter	Center of Floor	15	10	y

TABLE A.2 Tee-beam Fundamental Frequencies, 2 in. x 12 in. Number Two, No Holes

Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
North Quarter	North Quarter	0	20.9	y	North Quarter	North Quarter	10	11.9	y
North Quarter	North Quarter	0	20.9	y	North Quarter	North Quarter	10	12	y
Center of Floor	North Quarter	0	21	n	Center of Floor	North Quarter	10	12.6	y
Center of Floor	North Quarter	0	20.9	y	Center of Floor	North Quarter	10	12.8	y
South Quarter	North Quarter	0	20.9	y	South Quarter	North Quarter	10	12	y
South Quarter	North Quarter	0	21	y	South Quarter	North Quarter	10	12.1	y
North Quarter	Center of Floor	0	20.7	y	North Quarter	Center of Floor	10	12.3	y
North Quarter	Center of Floor	0	20.6	y	North Quarter	Center of Floor	10	12.4	y
Center of Floor	Center of Floor	0	21	n	Center of Floor	Center of Floor	10	12.8	n
Center of Floor	Center of Floor	0	20.5	n	Center of Floor	Center of Floor	10	12.7	n
South Quarter	Center of Floor	0	20.8	y	South Quarter	Center of Floor	10	12.4	y
South Quarter	Center of Floor	0	20.6	y	South Quarter	Center of Floor	10	12.5	y
North Quarter	South Quarter	0	21	y	North Quarter	South Quarter	10	12.3	y
North Quarter	South Quarter	0	20.9	y	North Quarter	South Quarter	10	12.3	y
Center of Floor	South Quarter	0	21	n	Center of Floor	South Quarter	10	12.6	y
Center of Floor	South Quarter	0	21	n	Center of Floor	South Quarter	10	12.9	y
South Quarter	South Quarter	0	20.7	y	South Quarter	South Quarter	10	12.1	y
South Quarter	South Quarter	0	20.9	n	South Quarter	South Quarter	10	12.1	y
North Quarter	North Quarter	5	14.7	y	North Quarter	North Quarter	15	10	n
North Quarter	North Quarter	5	14.6	y	North Quarter	North Quarter	15	10	n
Center of Floor	North Quarter	5	15.2	y	Center of Floor	North Quarter	15	10	n
Center of Floor	North Quarter	5	15	y	Center of Floor	North Quarter	15	10	n
South Quarter	North Quarter	5	15.1	y	South Quarter	North Quarter	15	10	n
South Quarter	North Quarter	5	14.9	y	South Quarter	North Quarter	15	11.3	n
North Quarter	Center of Floor	5	14.4	y	North Quarter	Center of Floor	15	11.3	y
North Quarter	Center of Floor	5	14.5	y	North Quarter	Center of Floor	15	11.2	y
Center of Floor	Center of Floor	5	14.3	n	Center of Floor	Center of Floor	15	10	n
Center of Floor	Center of Floor	5	14.4	y	Center of Floor	Center of Floor	15	10.3	y
South Quarter	Center of Floor	5	14.6	y	South Quarter	Center of Floor	15	11.2	y
South Quarter	Center of Floor	5	14.5	y	South Quarter	Center of Floor	15	10.4	y
North Quarter	South Quarter	5	14.6	y	North Quarter	South Quarter	15	11.2	y
North Quarter	South Quarter	5	14.5	y	North Quarter	South Quarter	15	11	n
Center of Floor	South Quarter	5	14.6	y	Center of Floor	South Quarter	15	11	y
Center of Floor	South Quarter	5	14.8	y	Center of Floor	South Quarter	15	11	n
South Quarter	South Quarter	5	9.6	y	South Quarter	South Quarter	15	10.8	y
South Quarter	South Quarter	5	14.6	n	South Quarter	South Quarter	15	10.6	y

TABLE A.2 Tee-beam Fundamental Frequencies, 2 in. x 12 in. Number Two, No Holes, Continued

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	18	n
at Wood Lab	Center of Floor	North Quarter	0	17	n
"	South Quarter	North Quarter	0	18	n
"	North Quarter	Center of Floor	0	17.5	n
"	Center of Floor	Center of Floor	0	17.5	n
"	South Quarter	Center of Floor	0	17.5	n
"	North Quarter	North Quarter	5	12.8	n
"	Center of Floor	North Quarter	5	14.4	n
"	South Quarter	North Quarter	5	12.8	y
"	North Quarter	Center of Floor	5	12.8	y
"	Center of Floor	Center of Floor	5	12.8	n
"	South Quarter	Center of Floor	5	12	n
"	North Quarter	North Quarter	10	9.7	y
"	Center of Floor	North Quarter	10	9.7	y
"	South Quarter	North Quarter	10	9.9	y
"	North Quarter	Center of Floor	10	10	y
"	Center of Floor	Center of Floor	10	9.8	y
"	South Quarter	Center of Floor	10	10	y

TABLE A.3 Tee-beam Fundamental Frequencies, Truss Number One

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	16.2	n
at Wood Lab	Center of Floor	North Quarter	0	12.8	y
"	South Quarter	North Quarter	0	16.8	n
"	North Quarter	Center of Floor	0	17	n
"	Center of Floor	Center of Floor	0	16	n
"	South Quarter	Center of Floor	0	16.4	y
"	North Quarter	North Quarter	5	12	y
"	Center of Floor	North Quarter	5	11.4	y
"	South Quarter	North Quarter	5	12	y
"	North Quarter	Center of Floor	5	12..5	y
"	Center of Floor	Center of Floor	5	12.8	y
"	South Quarter	Center of Floor	5	12.1	y
"	North Quarter	North Quarter	10	10.2	y
"	Center of Floor	North Quarter	10	10.2	y
"	South Quarter	North Quarter	10	10.3	y
"	North Quarter	Center of Floor	10	9.9	y
"	Center of Floor	Center of Floor	10	10	y
"	South Quarter	Center of Floor	10	10.1	y

TABLE A.3 Tee-beam Fundamental Frequencies, Truss Number One, Continued

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	Center of Floor	0	20.4	n
at Structures Lab	North Quarter	Center of Floor	0	20.7	n
on steel test frame	Center of Floor	Center of Floor	0	20.6	n
Steel sole plates	Center of Floor	Center of Floor	0	20.3	n
placed between	South Quarter	Center of Floor	0	20.7	n
frame and joists.	South Quarter	Center of Floor	0	20.3	n
"	North Quarter	Center of Floor	5	13.4	y
"	North Quarter	Center of Floor	5	13.9	y
"	Center of Floor	Center of Floor	5	12.2	n
"	Center of Floor	Center of Floor	5	14.3	y
"	South Quarter	Center of Floor	5	13.7	y
"	South Quarter	Center of Floor	5	13.8	y
"	North Quarter	Center of Floor	10	11	y
"	North Quarter	Center of Floor	10	11.2	y
"	Center of Floor	Center of Floor	10	10.5	y
"	Center of Floor	Center of Floor	10	10.6	y
"	South Quarter	Center of Floor	10	11	y
"	South Quarter	Center of Floor	10	11.1	y
"	North Quarter	Center of Floor	15	9.8	y
"	North Quarter	Center of Floor	15	9.8	y
"	Center of Floor	Center of Floor	15	9.1	y
"	Center of Floor	Center of Floor	15	9.5	y
"	South Quarter	Center of Floor	15	9.6	y
"	South Quarter	Center of Floor	15	9.7	y
Steel sole plates	North Quarter	North Quarter	5	13.3	y
were replaced with	North Quarter	North Quarter	5	13.5	y
with wood sill plates	Center of Floor	North Quarter	5	13.1	n
Data acquisition	Center of Floor	North Quarter	5	13.8	y
equipement NOT	South Quarter	North Quarter	5	13.3	y
placed on floor	South Quarter	North Quarter	5	13.5	y
"	North Quarter	Center of Floor	5	13.3	y
"	North Quarter	Center of Floor	5	13.1	y
"	Center of Floor	Center of Floor	5	13.4	n
"	Center of Floor	Center of Floor	5	13.8	n
"	South Quarter	Center of Floor	5	13.4	y
"	South Quarter	Center of Floor	5	13.7	y
"	North Quarter	North Quarter	10	11	y
"	North Quarter	North Quarter	10	10.9	y
"	Center of Floor	North Quarter	10	11.3	y
"	Center of Floor	North Quarter	10	10.7	y
"	South Quarter	North Quarter	10	11	y
"	South Quarter	North Quarter	10	11	y
"	North Quarter	Center of Floor	10	11.1	y
"	North Quarter	Center of Floor	10	10.9	y
"	Center of Floor	Center of Floor	10	11	y
"	Center of Floor	Center of Floor	10	11.3	n
"	South Quarter	Center of Floor	10	11	y
"	South Quarter	Center of Floor	10	11	n

TABLE A.4 Tee-beam Fundamental Frequencies, Truss Number Two

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	21.3	n
at Structures Lab	North Quarter	North Quarter	0	21.3	n
on steel test frame	Center of Floor	North Quarter	0	20.8	n
Steel sole plates	Center of Floor	North Quarter	0	21	n
placed between	South Quarter	North Quarter	0	18.2	y
frame and joists.	South Quarter	North Quarter	0	21.4	n
"	North Quarter	Center of Floor	0	18.5	y
"	North Quarter	Center of Floor	0	21.3	n
"	Center of Floor	Center of Floor	0	18.8	n
"	Center of Floor	Center of Floor	0	21.3	n
"	South Quarter	Center of Floor	0	21.2	n
"	South Quarter	Center of Floor	0	21.1	n
"	North Quarter	South Quarter	0	21.2	n
"	North Quarter	South Quarter	0	21.3	n
"	Center of Floor	South Quarter	0	21.4	n
"	Center of Floor	South Quarter	0	21.2	n
"	South Quarter	South Quarter	0	21.3	n
"	South Quarter	South Quarter	0	21.4	n
"	North Quarter	Center of Floor	5	14	y
"	North Quarter	Center of Floor	5	14.1	y
"	Center of Floor	Center of Floor	5	13.9	n
"	Center of Floor	Center of Floor	5	14.4	y
"	South Quarter	Center of Floor	5	14.2	y
"	South Quarter	Center of Floor	5	14.6	y
"	North Quarter	Center of Floor	10	11.3	y
"	North Quarter	Center of Floor	10	11.3	y
"	Center of Floor	Center of Floor	10	9.9	n
"	Center of Floor	Center of Floor	10	11.2	y
"	South Quarter	Center of Floor	10	11.3	y
"	South Quarter	Center of Floor	10	11.4	y
"	North Quarter	Center of Floor	15	9.9	y
"	North Quarter	Center of Floor	15	10.1	y
"	Center of Floor	Center of Floor	15	9.9	y
"	Center of Floor	Center of Floor	15	9.7	y
"	South Quarter	Center of Floor	15	9.9	y
"	South Quarter	Center of Floor	15	9.9	y

TABLE A.5 Four Joist Tee-beam Fundamental Frequencies, 8 ft. x 16 ft. 9.5 in. I-joist, With Holes

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	15.1	y
at Wood Lab	North Quarter	North Quarter	0	15.2	y
"	Center of Floor	North Quarter	0	15	n
"	Center of Floor	North Quarter	0	15	n
"	South Quarter	North Quarter	0	15	n
"	South Quarter	North Quarter	0	15.2	y
"	North Quarter	Center of Floor	0	14.7	y
"	North Quarter	Center of Floor	0	15	n
"	Center of Floor	Center of Floor	0	14.5	n
"	Center of Floor	Center of Floor	0	14	n
"	South Quarter	Center of Floor	0	14.8	y
"	South Quarter	Center of Floor	0	14.8	y
"	North Quarter	North Quarter	5	10	y
"	North Quarter	North Quarter	5	10.2	y
"	Center of Floor	North Quarter	5	10.1	y
"	Center of Floor	North Quarter	5	10.1	y
"	South Quarter	North Quarter	5	10.3	y
"	South Quarter	North Quarter	5	10.2	y
"	North Quarter	Center of Floor	5	error	error
"	North Quarter	Center of Floor	5	10.2	y
"	Center of Floor	Center of Floor	5	10.2	y
"	Center of Floor	Center of Floor	5	10	n
"	South Quarter	Center of Floor	5	10.1	y
"	South Quarter	Center of Floor	5	10.3	y
"	North Quarter	North Quarter	10	8.4	y
"	North Quarter	North Quarter	10	8.3	y
"	Center of Floor	North Quarter	10	8.3	y
"	Center of Floor	North Quarter	10	8.4	y
"	South Quarter	North Quarter	10	8.3	y
"	South Quarter	North Quarter	10	8.4	y
"	North Quarter	Center of Floor	10	8.3	y
"	North Quarter	Center of Floor	10	8.4	y
"	Center of Floor	Center of Floor	10	8.4	y
"	Center of Floor	Center of Floor	10	8.1	y
"	South Quarter	Center of Floor	10	8.4	y
"	South Quarter	Center of Floor	10	8.5	n

TABLE A.6 Tee-beam Fundamental Frequencies, 4 ft. x 16 ft. 9.5 in. I-joist, With Holes

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	14.8	y
at Wood Lab	North Quarter	North Quarter	0	15	y
"	Center of Floor	North Quarter	0	16.5	n
"	Center of Floor	North Quarter	0	15.7	n
"	South Quarter	North Quarter	0	15.1	y
"	South Quarter	North Quarter	0	15.5	y
"	North Quarter	Center of Floor	0	15	n
"	North Quarter	Center of Floor	0	14.9	y
"	Center of Floor	Center of Floor	0	14.4	n
"	Center of Floor	Center of Floor	0	14.9	n
"	South Quarter	Center of Floor	0	15	n
"	South Quarter	Center of Floor	0	14.8	y
"	North Quarter	North Quarter	5	9.4	y
"	North Quarter	North Quarter	5	9.5	y
"	Center of Floor	North Quarter	5	11	n
"	Center of Floor	North Quarter	5	8.9	y
"	South Quarter	North Quarter	5	9.9	y
"	South Quarter	North Quarter	5	9.9	n
"	North Quarter	Center of Floor	5	10.3	y
"	North Quarter	Center of Floor	5	10.3	y
"	Center of Floor	Center of Floor	5	9.9	y
"	Center of Floor	Center of Floor	5	9.1	y
"	South Quarter	Center of Floor	5	10.4	y
"	South Quarter	Center of Floor	5	10.3	y
"	North Quarter	North Quarter	10	8.3	y
"	North Quarter	North Quarter	10	8.2	y
"	Center of Floor	North Quarter	10	8.9	y
"	Center of Floor	North Quarter	10	8.8	y
"	South Quarter	North Quarter	10	8.5	y
"	South Quarter	North Quarter	10	8.6	y
"	North Quarter	Center of Floor	10	8.4	y
"	North Quarter	Center of Floor	10	8.3	y
"	Center of Floor	Center of Floor	10	9.4	n
"	Center of Floor	Center of Floor	10	8	y
"	South Quarter	Center of Floor	10	8.1	y
"	South Quarter	Center of Floor	10	7.9	y

TABLE A.6 Tee-beam Fundamental Frequencies, 4 ft. x 16 ft. 9.5 in. I-joist, With Holes, Continued

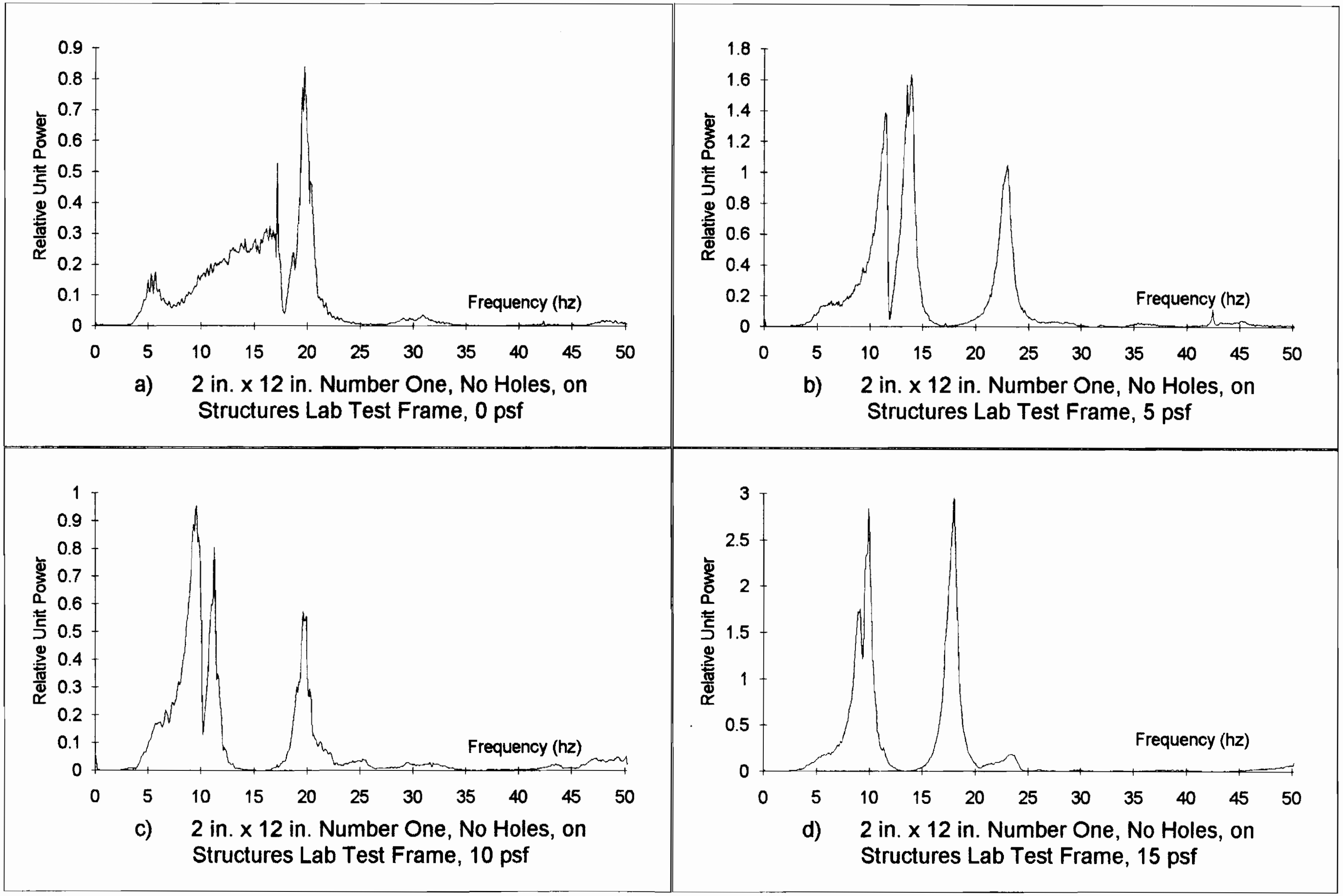
	Heel Drop Location	Accelerometer location	Dead load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed at Structures Lab on steel test frame	North Quarter	North Quarter	0	16.8	y
	Center of Floor	North Quarter	0	17.9	n
	South Quarter	North Quarter	0	17.2	n
"	North Quarter	Center of Floor	0	16.8	n
"	Center of Floor	Center of Floor	0	17.0	n
"	South Quarter	Center of Floor	0	16.9	n
"	North Quarter	North Quarter	5	10.4	y
"	Center of Floor	North Quarter	5	8.6	n
"	South Quarter	North Quarter	5	10.9	y
"	North Quarter	Center of Floor	5	10.5	y
"	Center of Floor	Center of Floor	5	11.2	n
"	South Quarter	Center of Floor	5	10.6	y
"	North Quarter	North Quarter	10	8.4	y
"	Center of Floor	North Quarter	10	8.3	y
"	South Quarter	North Quarter	10	8.3	y
"	North Quarter	Center of Floor	10	8.0	y
"	Center of Floor	Center of Floor	10	8.3	y
"	South Quarter	Center of Floor	10	8.7	y
Test performed at Structures Lab on concrete floor	North Quarter	North Quarter	0	17.4	y
	Center of Floor	North Quarter	0	17.9	y
	South Quarter	North Quarter	0	18.0	n
"	North Quarter	Center of Floor	0	17.1	n
"	Center of Floor	Center of Floor	0	17.4	n
"	South Quarter	Center of Floor	0	18.0	n
"	North Quarter	North Quarter	5	error	error
"	Center of Floor	North Quarter	5	error	error
"	South Quarter	North Quarter	5	10.0	y
"	North Quarter	Center of Floor	5	10.3	n
"	Center of Floor	Center of Floor	5	9.5	n
"	South Quarter	Center of Floor	5	10.2	y
"	North Quarter	North Quarter	10	7.9	y
"	Center of Floor	North Quarter	10	9.8	n
"	South Quarter	North Quarter	10	8.2	y
"	North Quarter	Center of Floor	10	8.3	y
"	Center of Floor	Center of Floor	10	8.6	n
"	South Quarter	Center of Floor	10	8.4	y

TABLE A.7 Tee-beam Fundamental Frequencies, Nominal 12 in. Large Flange I-joist, With Holes

	Heel Drop Location	Accelerometer Location	Dead Load (psf)	Fundamental Frequency (Hz)	Was this the first significant and largest spike?
Test performed	North Quarter	North Quarter	0	21.3	y
at Structures Lab	North Quarter	North Quarter	0	21.2	n
on steel test frame	Center of Floor	North Quarter	0	21.9	n
Headers on steel	Center of Floor	North Quarter	0	21	n
sole plates and are					
clamped to frame	North Quarter	Center of Floor	0	21.5	y
"	North Quarter	Center of Floor	0	21.2	y
"	Center of Floor	Center of Floor	0	21	n
"	Center of Floor	Center of Floor	0	21	n
"	North Quarter	North Quarter	5	14.3	y
"	North Quarter	North Quarter	5	14.4	y
"	Center of Floor	North Quarter	5	14.9	n
"	Center of Floor	North Quarter	5	14.5	y
"	North Quarter	Center of Floor	5	14.4	y
"	North Quarter	Center of Floor	5	14.4	y
"	Center of Floor	Center of Floor	5	14.4	y
"	Center of Floor	Center of Floor	5	14.3	y
"	North Quarter	North Quarter	10	11.8	y
"	North Quarter	North Quarter	10	11.7	y
"	Center of Floor	North Quarter	10	12.3	y
"	Center of Floor	North Quarter	10	12.1	y
"	North Quarter	Center of Floor	10	11.9	y
"	North Quarter	Center of Floor	10	11.8	y
"	Center of Floor	Center of Floor	10	12	y
"	Center of Floor	Center of Floor	10	12	y
"	North Quarter	North Quarter	15	11.1	n
"	North Quarter	North Quarter	15	10.5	y
"	Center of Floor	North Quarter	15	11.1	n
"	Center of Floor	North Quarter	15	11	y
"	North Quarter	Center of Floor	15	10.9	n
"	North Quarter	Center of Floor	15	11	y
"	Center of Floor	Center of Floor	15	10.9	y
"	Center of Floor	Center of Floor	15	10.9	y
"	North Quarter	North Quarter	20	9.4	y
"	North Quarter	North Quarter	20	9.4	y
"	Center of Floor	North Quarter	20	10.1	n
"	Center of Floor	North Quarter	20	9.5	y
"	North Quarter	Center of Floor	20	9.5	y
"	North Quarter	Center of Floor	20	9.4	y
"	Center of Floor	Center of Floor	20	9.11	y
"	Center of Floor	Center of Floor	20	9.21	y

APPENDIX B

Tee-Beam Power Spectrums



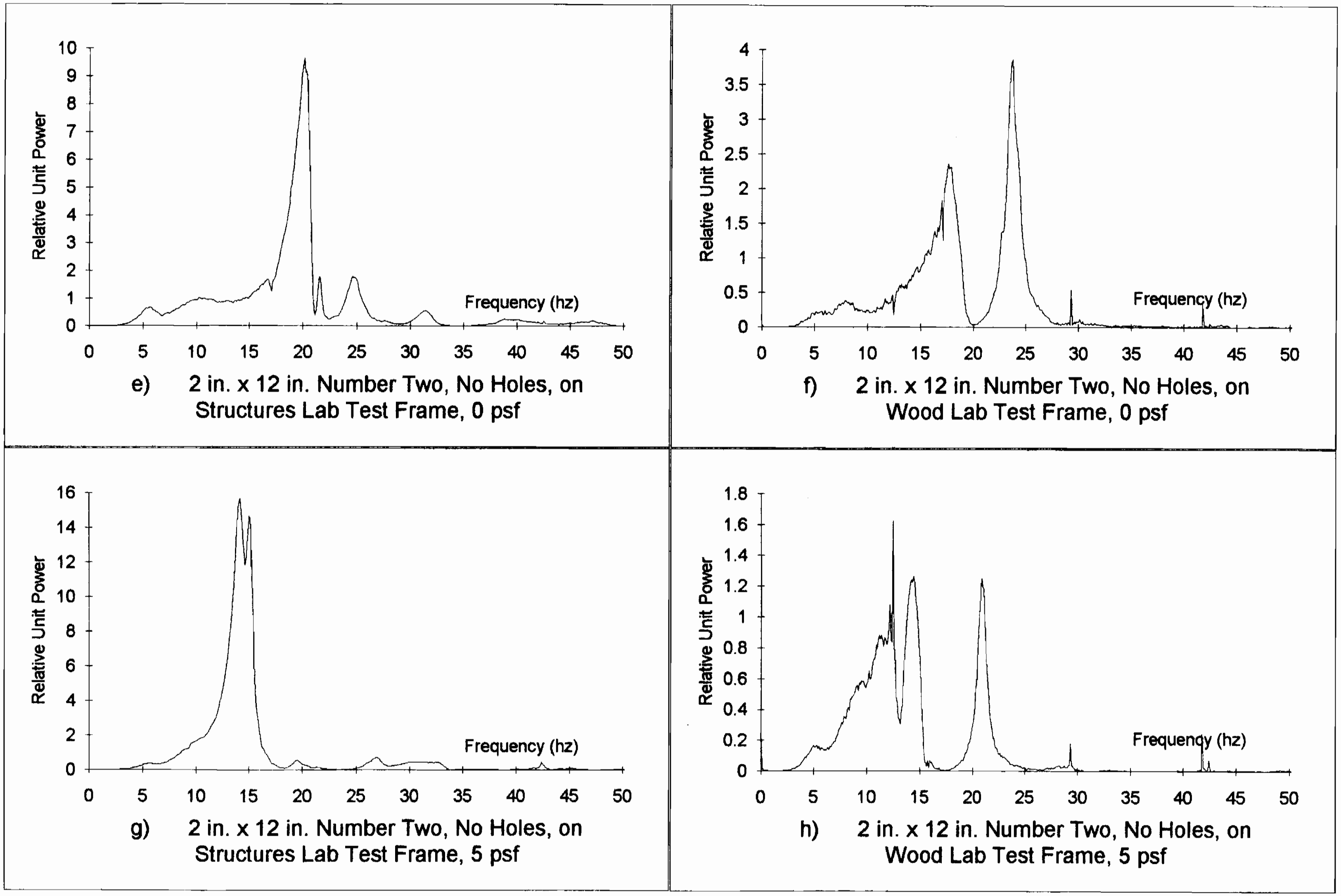
a) 2 in. x 12 in. Number One, No Holes, on Structures Lab Test Frame, 0 psf

b) 2 in. x 12 in. Number One, No Holes, on Structures Lab Test Frame, 5 psf

c) 2 in. x 12 in. Number One, No Holes, on Structures Lab Test Frame, 10 psf

d) 2 in. x 12 in. Number One, No Holes, on Structures Lab Test Frame, 15 psf

FIGURE B.1 Relative Unit Power versus Frequency, Solid-Sawn Lumber Joists



e) 2 in. x 12 in. Number Two, No Holes, on Structures Lab Test Frame, 0 psf

f) 2 in. x 12 in. Number Two, No Holes, on Wood Lab Test Frame, 0 psf

g) 2 in. x 12 in. Number Two, No Holes, on Structures Lab Test Frame, 5 psf

h) 2 in. x 12 in. Number Two, No Holes, on Wood Lab Test Frame, 5 psf

FIGURE B.1 Relative Unit Power versus Frequency, Solid-Sawn Lumber Joists, Continued

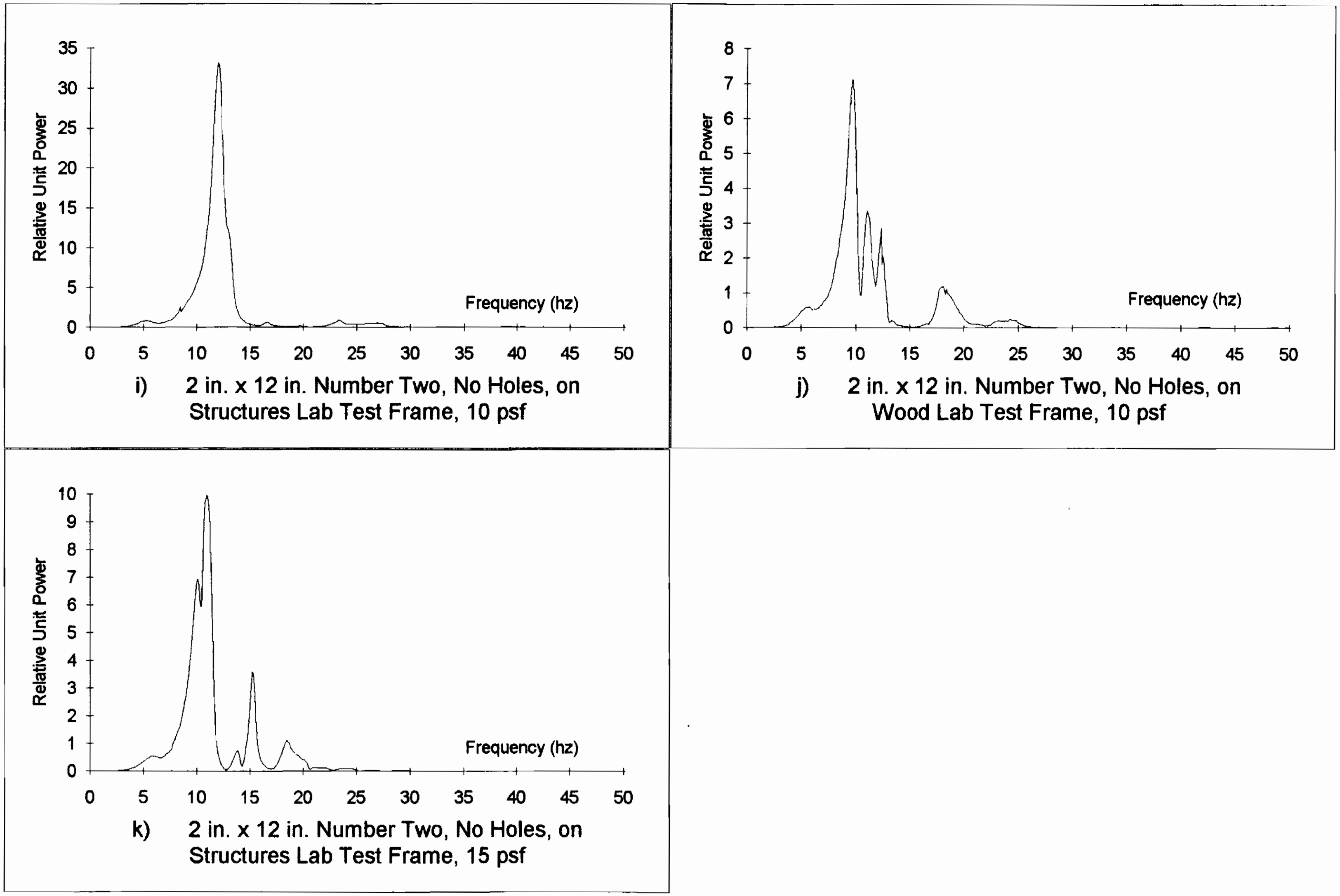
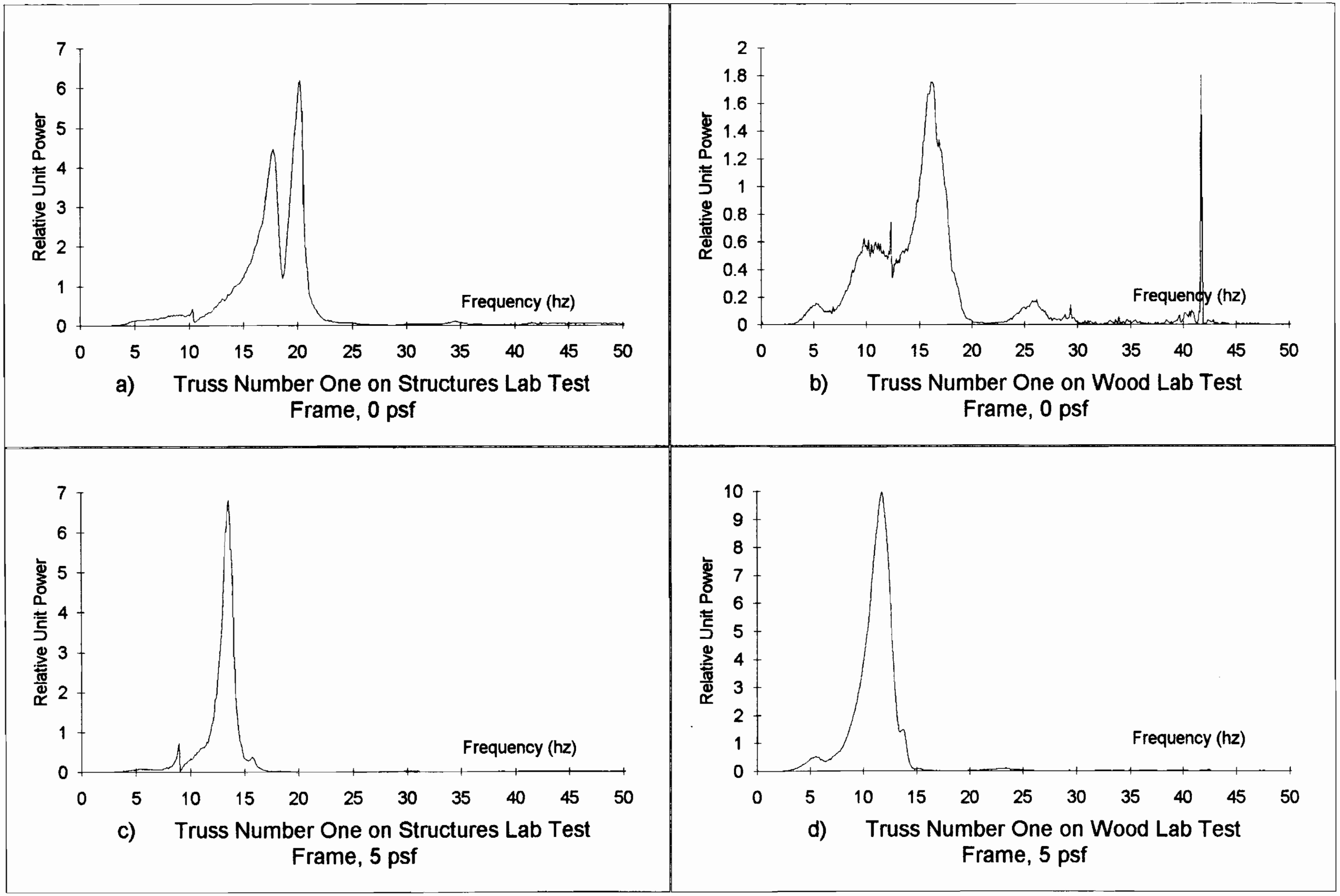


FIGURE B.1 Relative Unit Power versus Frequency, Solid-Sawn Lumber Joists, Continued



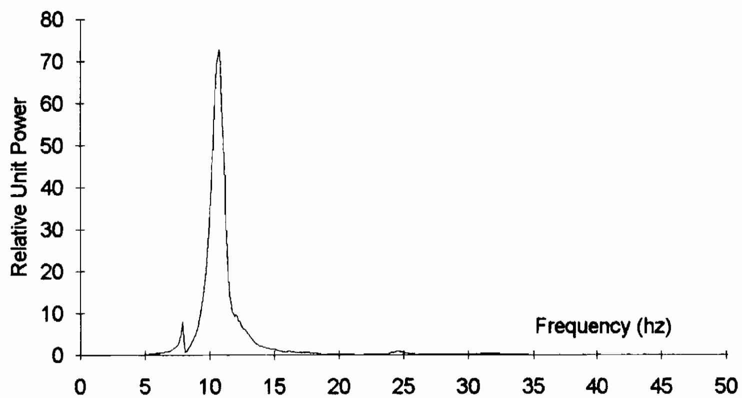
a) Truss Number One on Structures Lab Test Frame, 0 psf

b) Truss Number One on Wood Lab Test Frame, 0 psf

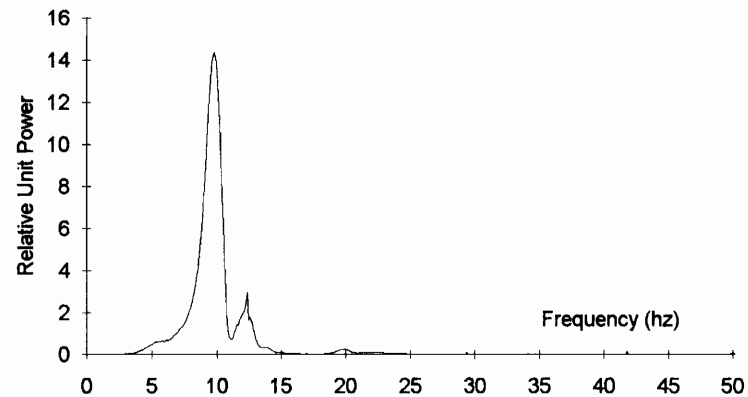
c) Truss Number One on Structures Lab Test Frame, 5 psf

d) Truss Number One on Wood Lab Test Frame, 5 psf

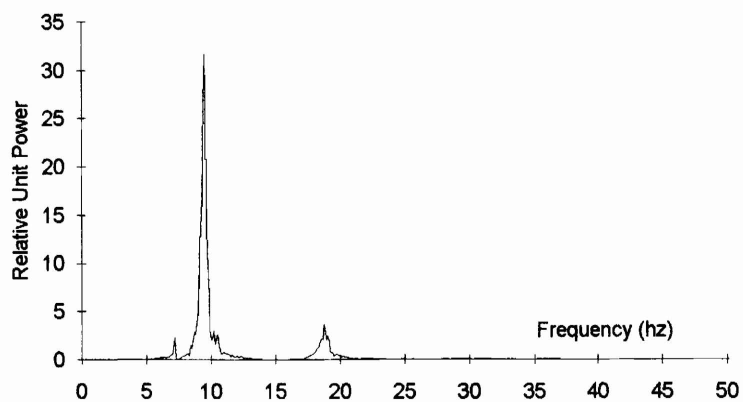
FIGURE B.2 Relative Unit Power versus Frequency, Truss Joists



e) Truss Number One on Structures Lab Test Frame, 10 psf

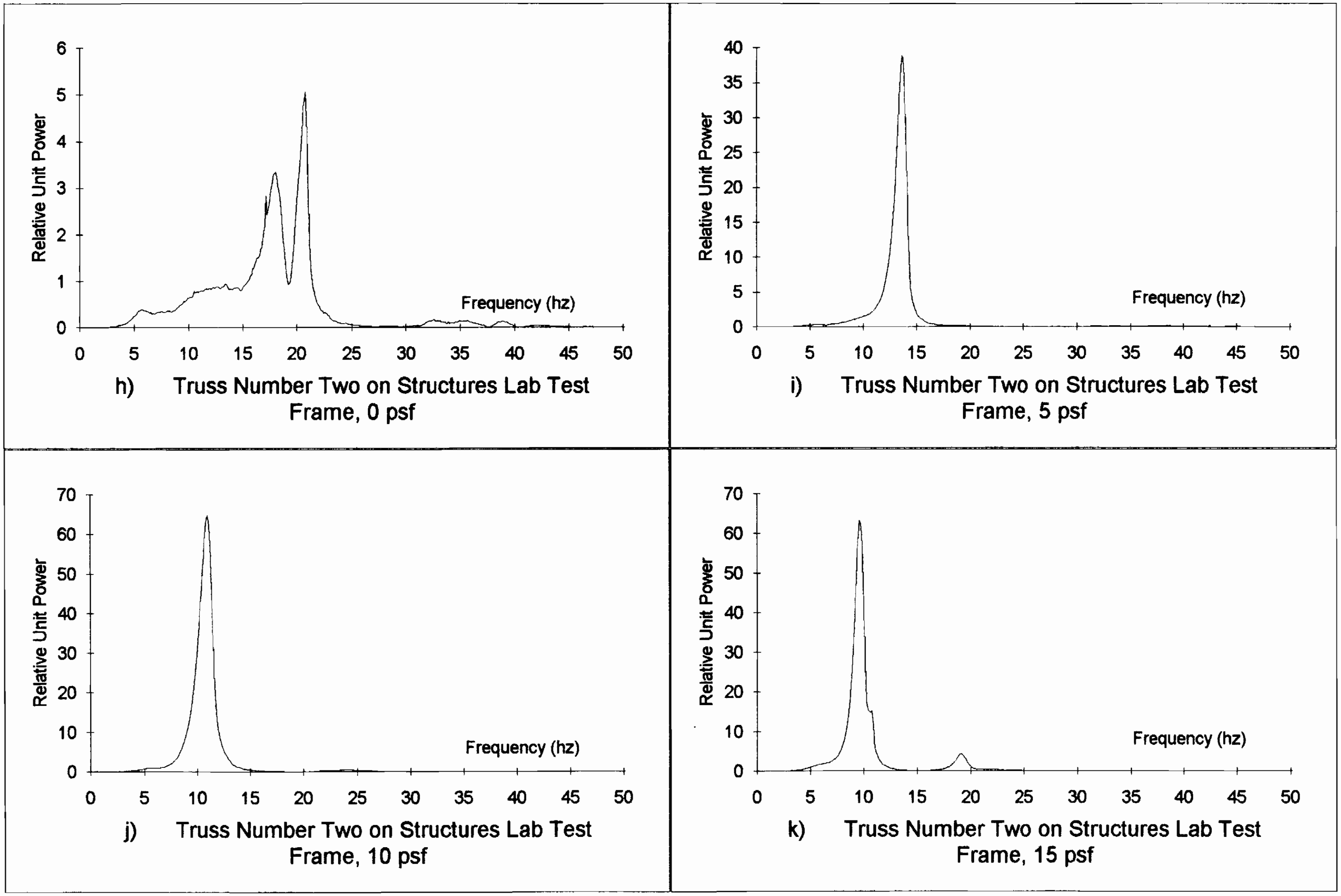


f) Truss Number One on Wood Lab Test Frame, 10 psf



g) Truss Number One on Structures Lab Test Frame, 15 psf

FIGURE B.2 Relative Unit Power versus Frequency, Truss Joists, Continued



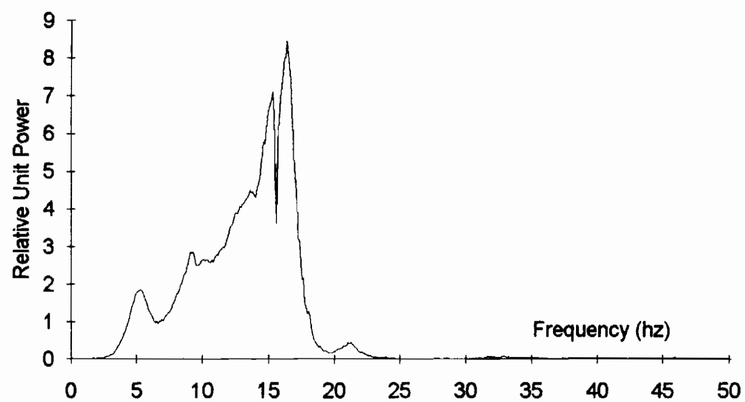
h) Truss Number Two on Structures Lab Test Frame, 0 psf

i) Truss Number Two on Structures Lab Test Frame, 5 psf

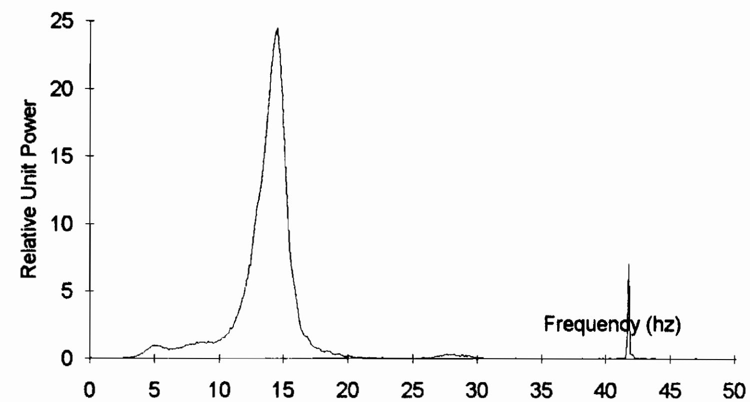
j) Truss Number Two on Structures Lab Test Frame, 10 psf

k) Truss Number Two on Structures Lab Test Frame, 15 psf

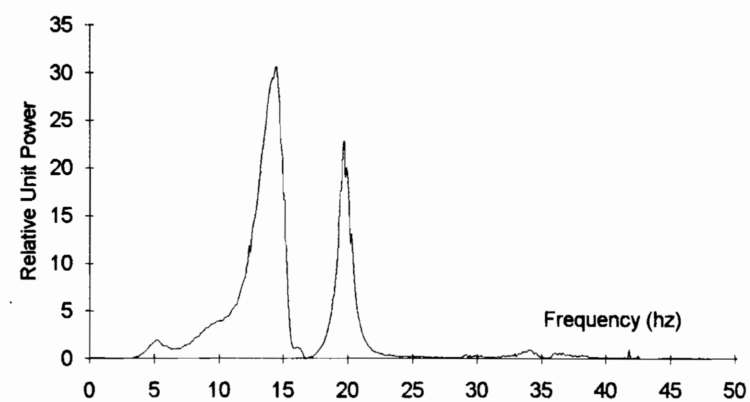
FIGURE B.2 Relative Unit Power versus Frequency, Truss Joists, Continued



a) 9.5 in. Composite I-joist,With Holes, on Structures Lab Test Frame, 0 psf

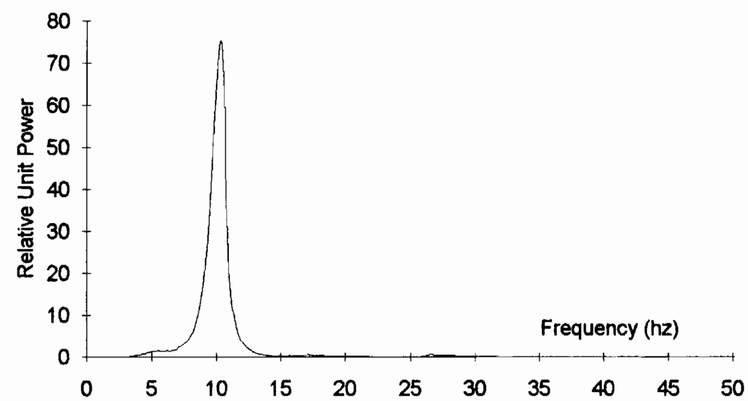


b) 9.5 in. Composite I-joist, With Holes, on Wood Lab Test Frame, 0 psf

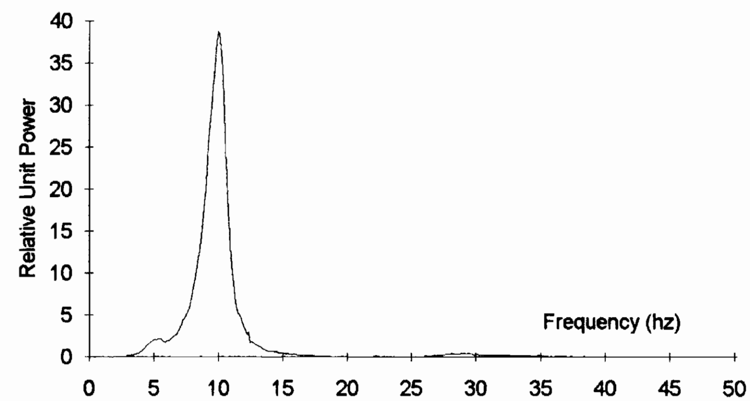


c) 9.5 in. Composite I-joist, With Holes, 8 ft. x 16 ft. specimen, on Wood Lab Test Frame, 0 psf

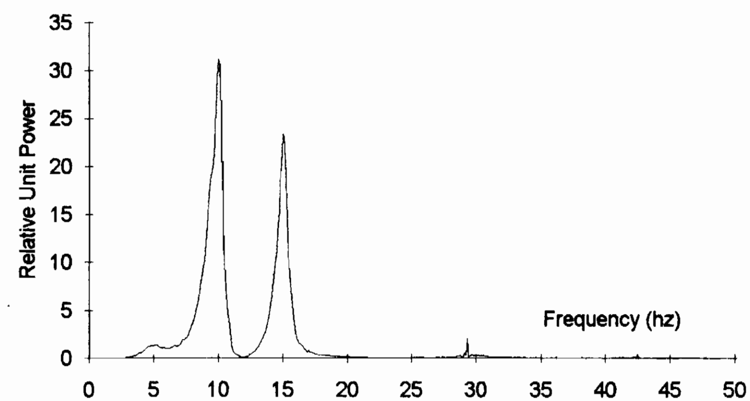
FIGURE B.3 Relative Unit Power versus Frequency, Composite I-joists



d) 9.5 in. Composite I-joist, With Holes, on Structures Lab Test Frame, 5 psf

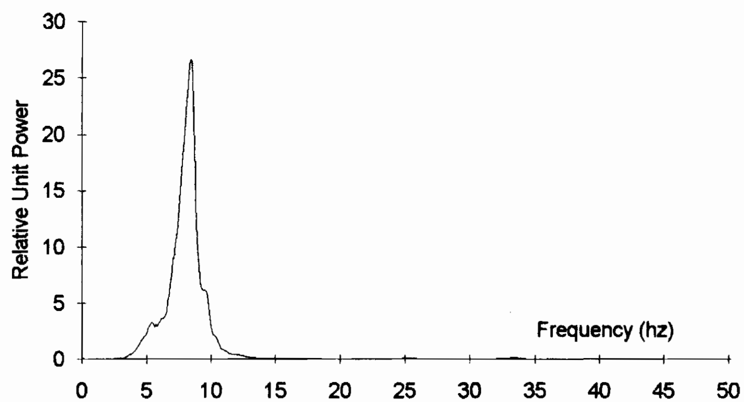


e) 9.5 in. Composite I-joist, With Holes, on Wood Lab Test Frame, 5 psf

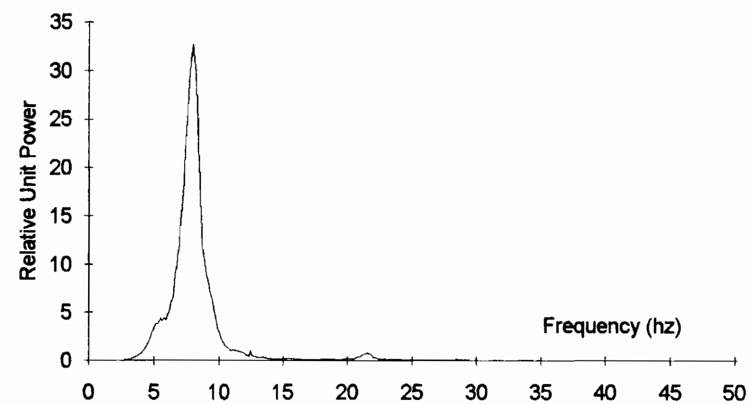


f) 9.5 in. Composite I-joist, With Holes, 8 ft. x 16 ft. specimen, on Wood Lab Test Frame, 5 psf

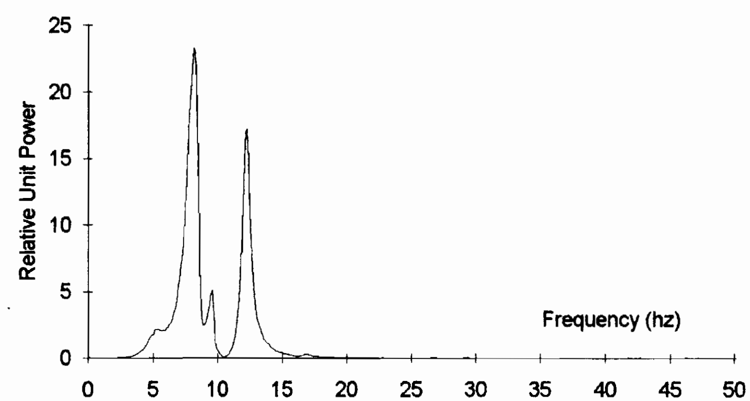
FIGURE B.3 Relative Unit Power versus Frequency, Composite I-joists, Continued



g) 9.5 in. Composite I-joist, With Holes, on Structures Lab Test Frame, 10 psf

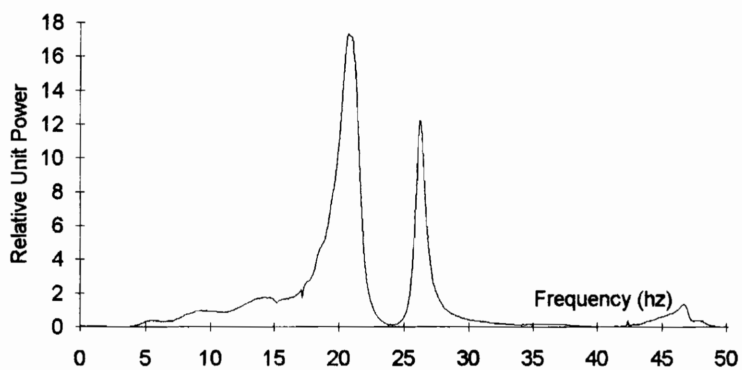


h) 9.5 in. Composite I-joist, With Holes, on Wood Lab Test Frame, 10 psf

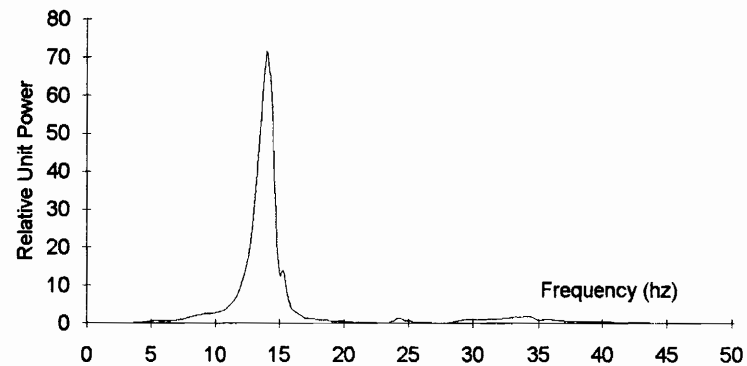


i) 9.5 in. Composite I-joist, With Holes, 8 ft. x 16 ft. specimen, on Wood Lab Test Frame, 10 psf

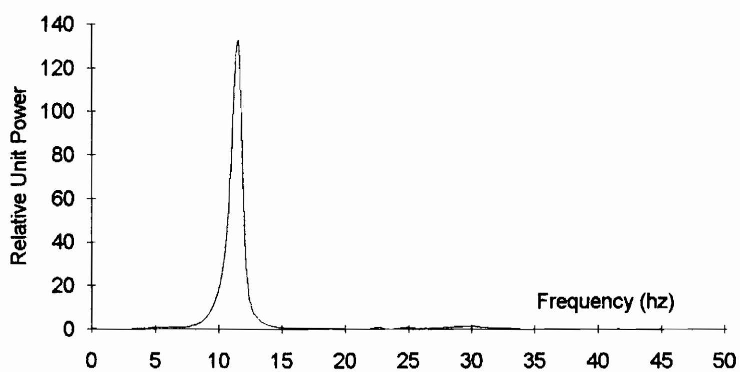
FIGURE B.3 Relative Unit Power versus Frequency, Composite I-joists, Continued



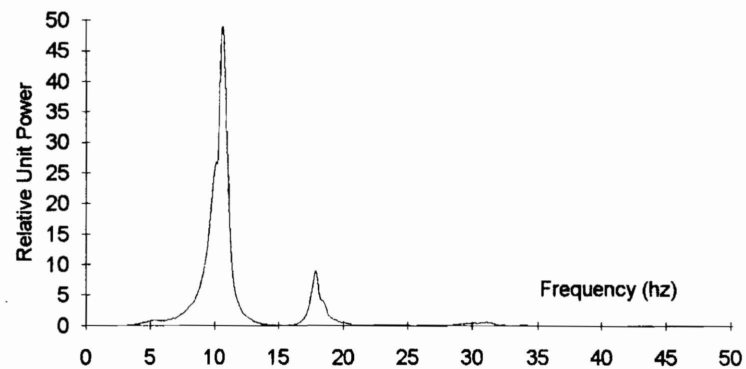
j) Nominal 12 in. Large Flange Composite I-joist, With Holes, on Structures Lab Test Frame, 0 psf



k) Nominal 12 in. Large Flange Composite I-joist, With Holes, on Structures Lab Test Frame, 5 psf



l) Nominal 12 in. Large Flange Compostite I-joist, With Holes, on Structures Lab Test Frame, 10 psf



m) Nominal 12 in. Large Flange Composite I-joist, With Holes, on Structures Lab Test Frame, 15 psf

FIGURE B.3 Relative Unit Power versus Frequency, Composite I-joists, Continued

VITA

David E. Runte was born in Omaha, Nebraska on May 7, 1968. He graduated from Ralston High School in Ralston, Nebraska in 1986. In 1990 he received his Bachelor of Science degree in Civil Engineering, with Distinction, from the University of Nebraska. He was employed at Enron Corporation/Northern Plains Natural Gas Company until entering the graduate program at VPI & SU in August, 1991.