EVALUATION OF A PROPOSED VIBRATION CRITERION

by

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(ABSTRACT)

A lightweight floor system consisting of a concrete slab on steel joists or beams is susceptible to annoying vibrations caused by walking. This investigation considers a criterion developed by Allen (1991) that indicates whether a proposed structural floor system is acceptable for walking vibrations.

The proposed vibration criterion is evaluated using the Murray vibration criterion as a basis for comparison. Both criteria are used to determine the acceptability of existing office floors, shopping malls, and pedestrian bridges. The evaluation results for each criteria are compared and the strengths and weaknesses of both criteria are discussed in detail.

A derivation is presented for the proposed criterion and the calculations involved in utilizing the criterion are described. Finally, recommendations for future research are discussed based on the evaluation results of the investigation.

ACKNOWLEDGEMENTS

I would like to thank Dr. Thomas M. Murray for pointing me in the direction of a thesis topic and giving sound advice during the writing of this work. I would also like to thank Dr. Easterling and Dr. Holzer for their input concerning the final product.

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CHAPTER I

INTRODUCTION AND LITERATURE SURVEY

1.1 Introduction

With the advent of lighter structural members and a drive toward lighter, less expensive floor systems, floor vibration problems have become prevalent in today's buildings. This is particularly true of floors with steel joist- or steel beam-concrete slab systems. Typically, it is very costly to solve a vibration problem in an existing building. It may involve adding weight to the floor (thus lowering the live load rating), experimenting with tuned mass dampers, or even placing supports under members to increase their stiffness. Although none of these repairs are guaranteed problem solvers, they are certainly expensive. The best solution is to design a floor system that will not result in vibration complaints. The goals of this study are to (1) evaluate existing vibration criteria and (2) evaluate a proposed criterion which improves upon the shortcomings found in the existing vibration criteria.

1.2 Terminology

Amplitude is the displacement of a point on a floor from its static equilibrium position. Acceleration, usually expressed as a percentage of acceleration due to gravity, is often substituted for amplitude when describing the motion of a floor. This substitution can be made since acceleration is simply the second derivative of displacement, or amplitude. In the case of transient vibration, maximum initial amplitude is the value of greatest interest. This is because the maximum initial amplitude is most noticed by floor occupants and successively smaller floor displacements are less readily perceived.

Damping is a measure of how rapidly a floor vibration decays. It can be described by the logarithmic decrement, which is the logarithm of the ratio of any two amplitudes, or accelerations (Figure 1.1). For greater accuracy, the ratio's numerator and denominator can be the average of the absolute value of successive positive and negative values of amplitude, or acceleration. Small changes in successive peak values indicate low damping for a floor, while large differences in peaks indicate high damping. Expressed in terms of the logarithmic decrement, damping is the ratio of viscous damping to critical damping. Critical damping is the minimum damping value at which a floor displaces from its equilibrium position and returns to that position without oscillation. Typically, a floor will possess 1.5% (bare floor) to 4.5% (finished floor without partitions) of critical damping.

Natural frequency is measured in cycles per second (hz). A cycle represents the smallest repeating section of a wave form which results from measuring the amplitude, or acceleration, of a point on a floor with respect to time. This cyclical portion of a wave form extends, for example, from one positive peak to the next positive peak. The number of cycles that occur in one second is the natural frequency and the amount of time required for one cycle to occur is known as the period (Figure 1.2).

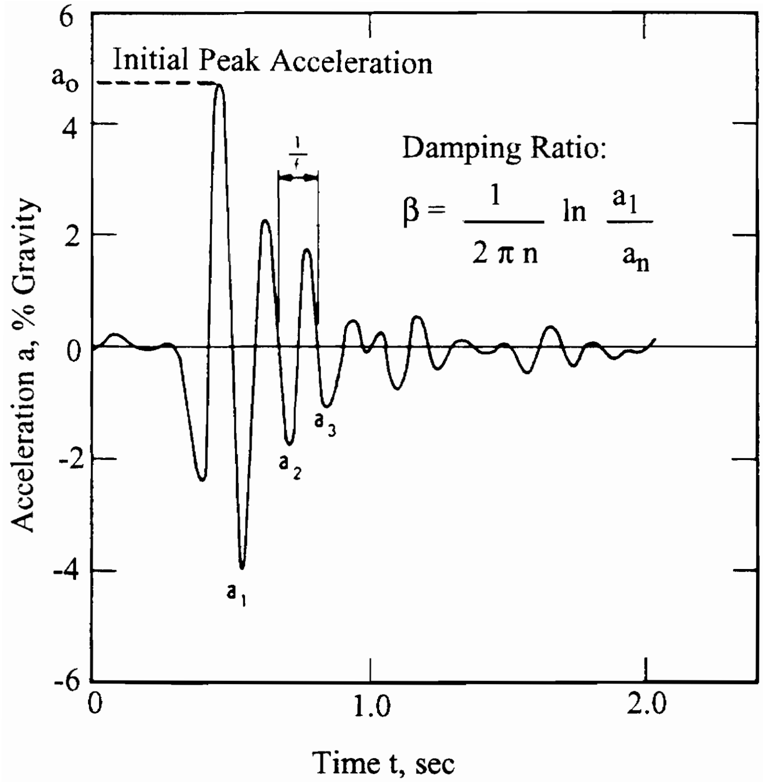
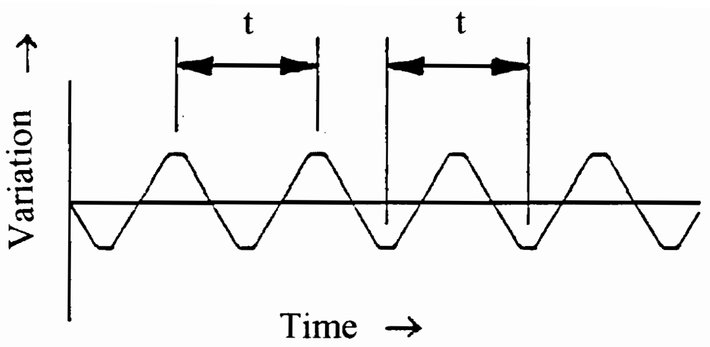


Figure 1.1 Definition of Damping [Allen 1976]



t = period (seconds)

f = 1/t = frequency (hz)

Figure 1.2 Definition of Frequency [Queen 1991]

Humans are especially sensitive to vibrations with frequencies in the range of 5 - 8 hz because certain body organs have similar frequencies, thereby causing resonance. Resonance is said to take place when an object achieves greater displacements after each cycle of vibration due to a forcing function with the same natural frequency, or a multiple of that frequency, known as a harmonic.

1.3 Literature Survey

The first half of the following literature survey is not in chronological order. This is because the work of Wiss and Parmelee (1974) was very similar to that of Reiher and Meister (Lenzen 1966). The two studies are directly compared in the first part of the literature survey by presenting Wiss and Parmelee's work immediately following Reiher and Meister's research.

The first noteworthy research concerning human response to floor vibration was conducted by Reiher and Meister in the early 1930's (Lenzen 1966). In their study, they subjected individuals to steady-state vibration. The people ranged from 20 to 37 years old and were tested in standing and reclining positions. Vibration frequencies ranged from 5 to 70 hz and amplitude varied from 0.001 to 0.04 in. Each person was exposed to vibration for a five minute period, and was asked to classify the vibration as slightly perceptible, distinctly perceptible, strongly perceptible, disturbing, or very disturbing. The results of the Reiher and Meister study for an individual standing and subjected to steady-state vertical vibration are shown in Figure 1.3. Only frequency and amplitude had to be varied for this research since the people were subjected to steady-state vibration rather than transient vibration.

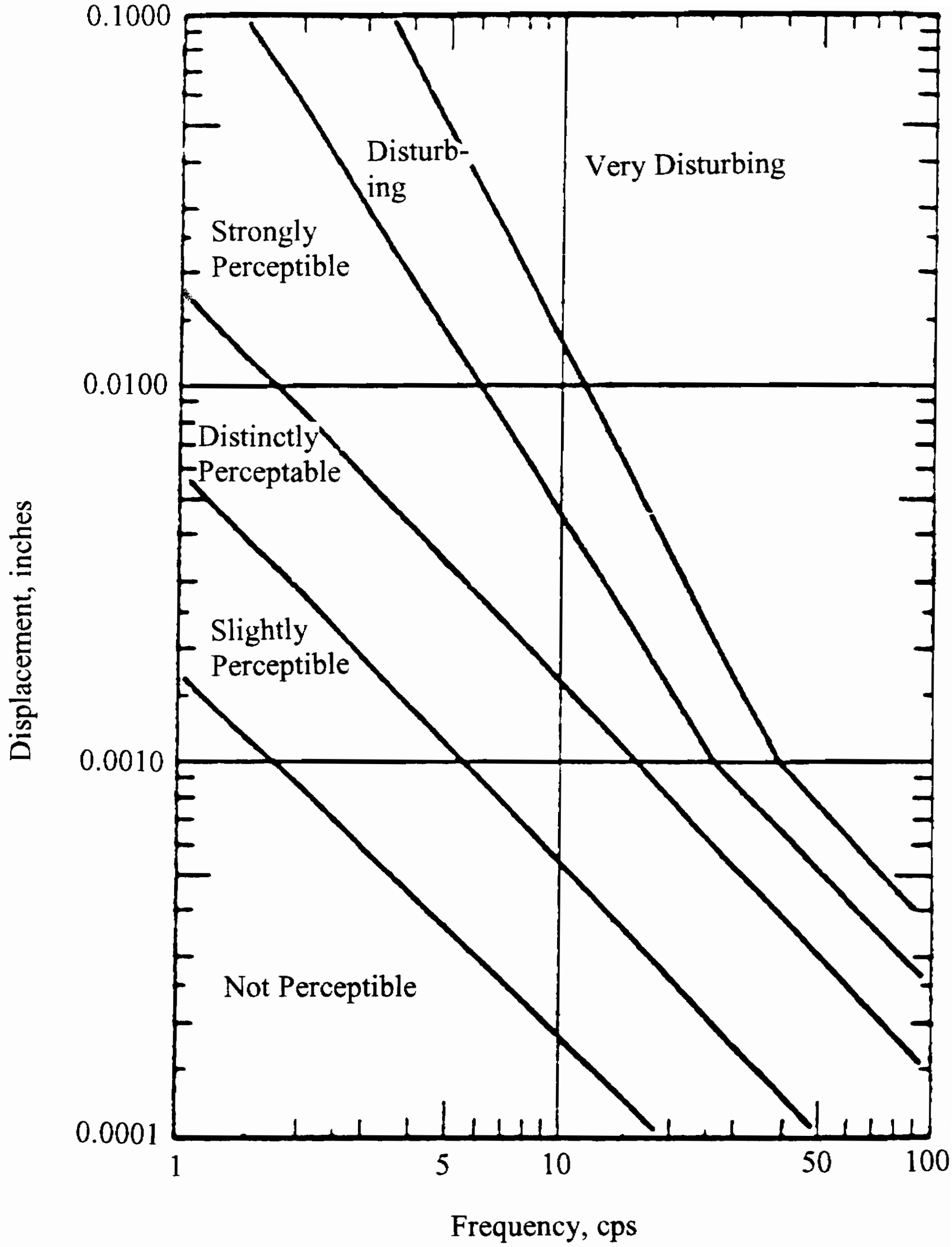


Figure 1.3 Reiher-Meister Scale [Wiss and Parmelee 1974]

Wiss and Parmelee (1974) studied the effect of transient floor vibrations on human beings. While Reiher and Meister only adjusted frequency and amplitude during their research, Wiss and Parmelee also varied damping to account for the transient nature of the vibration. Each of the 40 people tested was asked to classify the vibration as imperceptible, barely perceptible, strongly perceptible, or severe. The subjects were placed on an hydraulic shaker that imitated actual floor vibrations caused by one foot fall. The resulting wave form built up to a maximum amplitude after a few cycles and then decayed with time, thus representing the damping that would be present in an actual structural system. Combinations of the following parameters were used in programming the shaker:

Frequencies:	2.5, 4, 6, 9, 14, 25 hz
Peak Displacements:	0.0001, 0.0003, 0.001, 0.003, 0.01, 0.03, 0.1 in.
Damping:	1, 2, 4, 8, 16 % of critical

Their research resulted in Figure 1.4 and the following prediction equation for damped vibration:

$$R = 5.08(fA / D^{0.217})^{0.265} \tag{1.1}$$

where R = response rating with R = 2 being barely perceptible, R=3 is distinctly perceptible and R = 4 is strongly perceptible; f = frequency, hz; A = maximum amplitude, in.; D = damping ratio.

By including zero damping, or steady-state vibration, Wiss and Parmelee (1974) made a comparison between their investigation and that of Reiher and Meister (Figure 1.5). The following equation was found for zero damping (Wiss

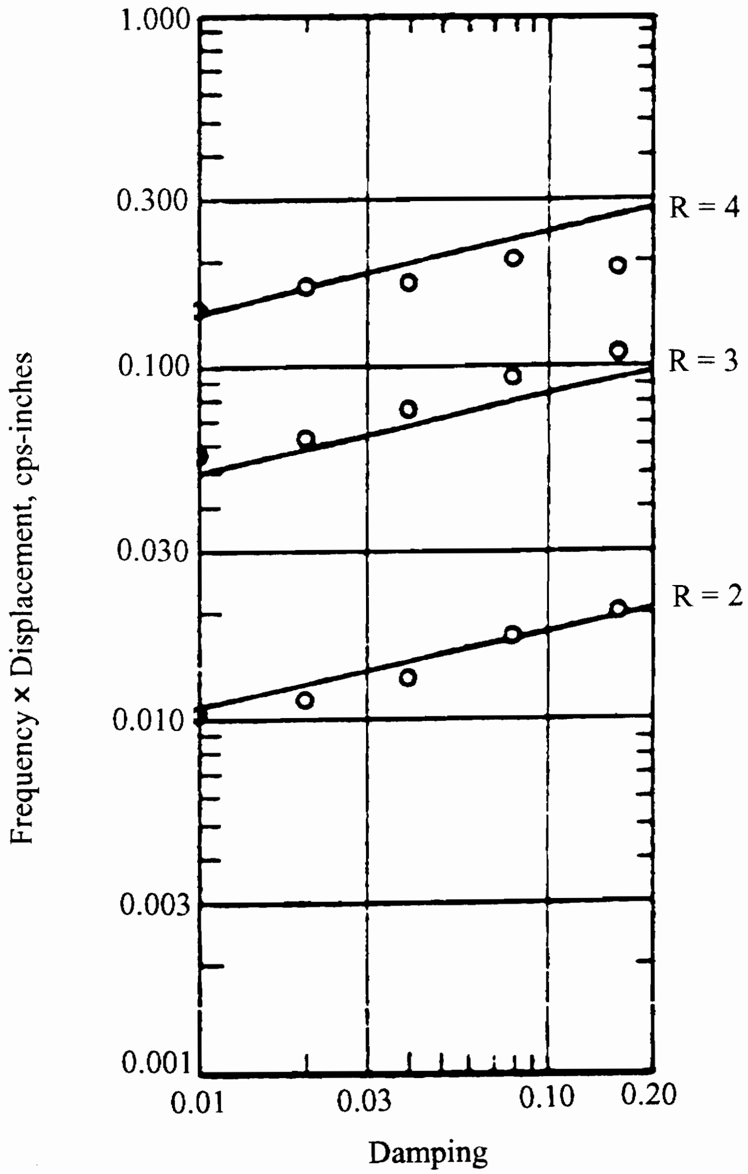


Figure 1.4 Wiss-Parmelee Scale [Wiss and Parmelee 1974]

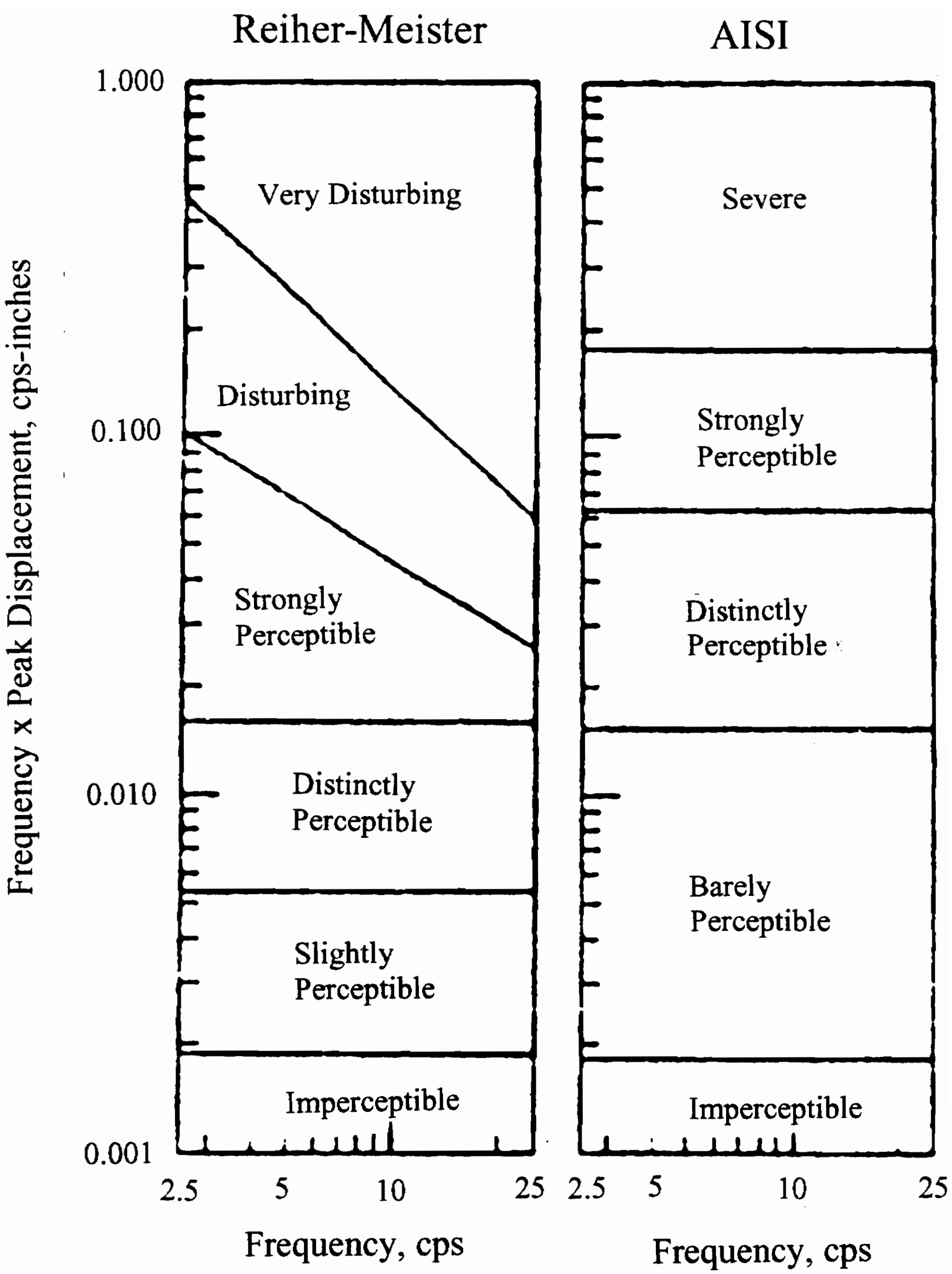


Figure 1.5 Comparison of Reiher-Meister and Wiss-Parmelee Scales
[Wiss and Parmelee 1974]

and Parmelee 1974):

$$R = 6.82(fA)^{0.24} \tag{1.2}$$

The lower limits of the two investigations were identical for the barely perceptible region but were higher for the other regions in Wiss and Parmelee's study. One reason for this difference is the duration of the vibration, which was only five seconds for Wiss and Parmelee's investigation as opposed to five minutes for Reiher and Meister's study. A long lasting vibration would tend to be more annoying than one of shorter duration. Reiher and Meister's lower limits for perceptibility might also be the result of setting the boundaries at the lowest combination of frequency and amplitude for a particular region. On the other hand, Wiss and Parmelee (1974) established the limits halfway between the mean values of frequency and amplitude for the different regions.

Standing or sitting seemed to have little effect on the subjects' responses to vibrations. In addition, age or sex seemed to make little difference on the results. Physiology, psychology, subjects' activity, or environment were not taken into account when developing the rating system. Possible resonant response of human body parts was also neglected. Wiss and Parmelee (1974) indicated that their findings only applied to floor vibrations having the same wave form characteristics as those found in Figure 1.6 and might not apply to floor vibration caused by walking.

Lenzen (1966) developed the modified Reiher-Meister scale by multiplying the amplitude of the original Reiher and Meister scale by a factor of 10 (Figure 1.7). He adjusted this scale to account for a human's greater sensitivity to steady-

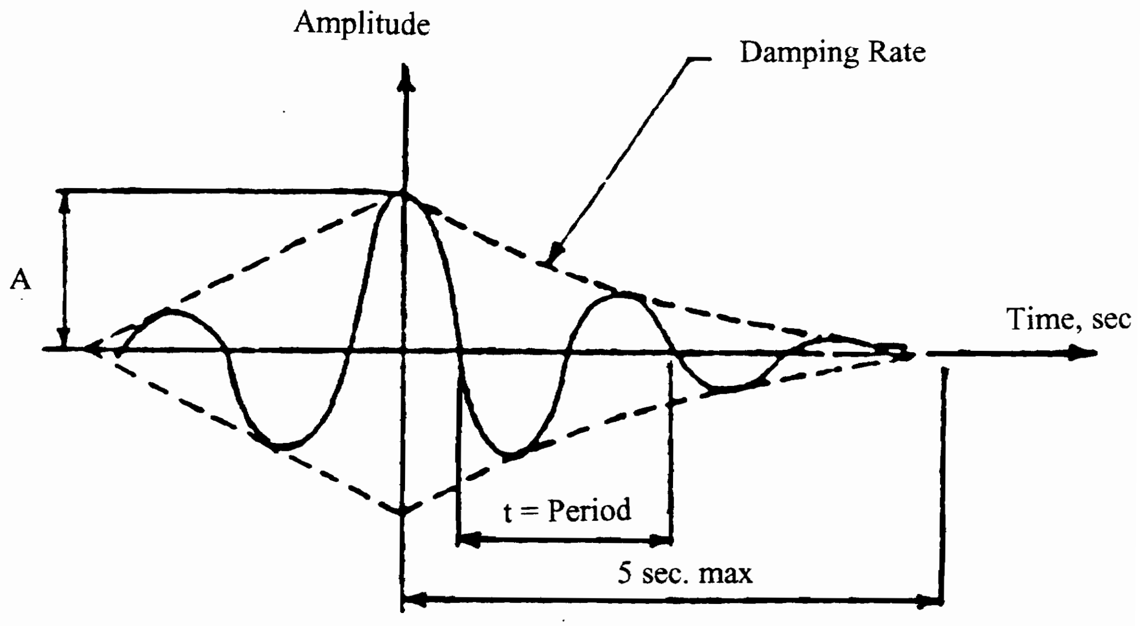


Figure 1.6 Wave Form of Vibration Signal [Wiss and Parmelee 1974]

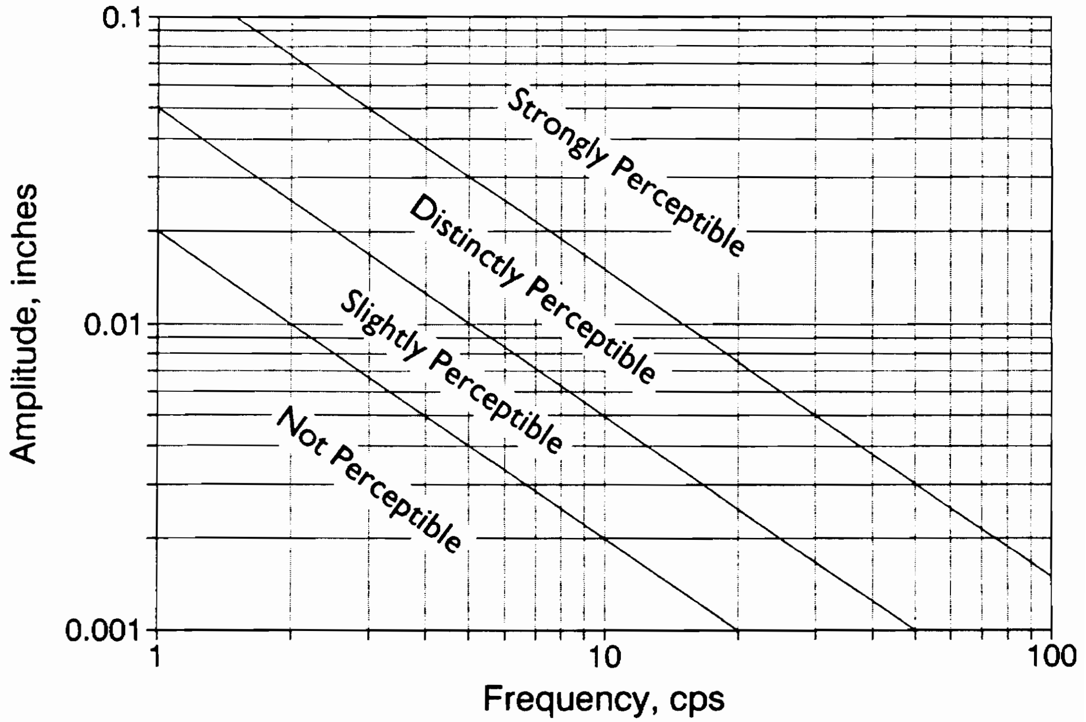


Figure 1.7 Modified Reiher-Meister Scale [Lenzen 1966]

state vibration than transient motion. Furthermore, he discovered that if a floor was damped to a small amplitude after five cycles, then only the impact was felt. This instance of rapid vibration decay corresponded to more than 5% damping. If the floor vibration lasted beyond 12 cycles, then the human response was the same as if it were a steady-state vibration. Based on these findings, Lenzen concluded that damping was more important than amplitude and frequency in determining a human's perception of vibration and he stated that the modified Reiher-Meister scale was applicable to floors with less than 5% damping.

The Canadian Standards Association (CSA) scale, which is based on the work of Allen and Rainer (1976), establishes peak acceleration limits for floors with certain frequency and damping characteristics (Figure 1.8). The heel impact test was used to measure the dynamic characteristics of long span floors. Next, these vibration properties were used in conjunction with subjective evaluation in order to develop the performance criterion. Limits for 3, 6 and 12% damping were developed by shifting the continuous vibration curve upward. The standard for 6% damping agreed with Lenzen's (1966) modification of the Reiher-Meister scale for floor systems without partitions. According to Allen and Rainer, the criterion was not applicable to short span floors because the one initiating and the one sensing the vibration act as dampers, causing the vibration to dissipate more rapidly. Instead of the transient vibration, the amplitude caused by walking was thought to have a greater influence on human response. Further, the CSA criterion pertain to quiet human occupancies such as residences, offices, and classrooms.

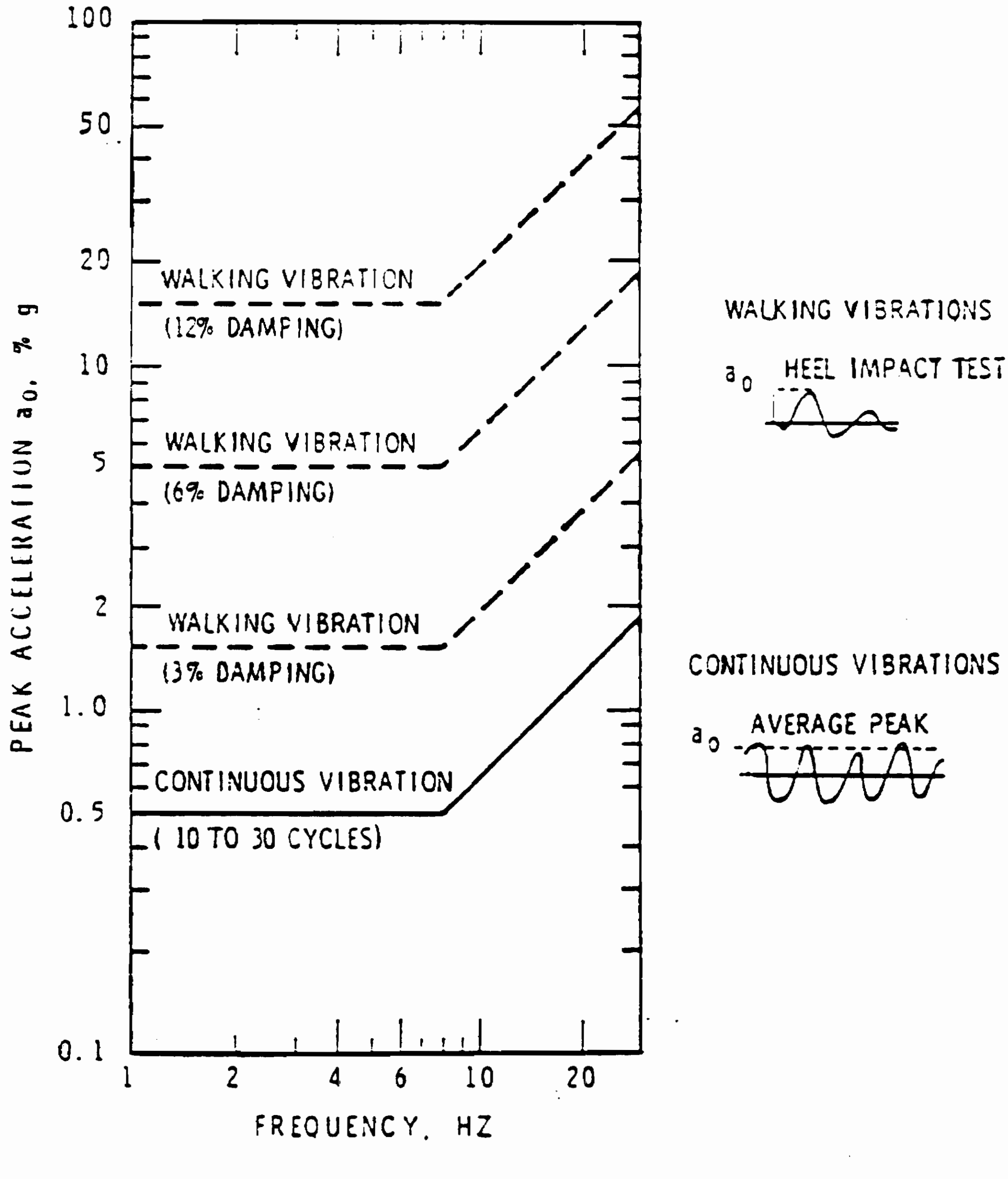


Figure 1.8 CSA Scale [Allen 1976]

Murray (1975) presented a design criterion for preventing floor vibrations. He demonstrated a method for calculating frequency and peak amplitude and then plotting the result on the modified Reiher-Meister scale developed by Lenzen (1966). Based on measurements and subjective evaluation, Murray concluded that a floor system which plotted in the upper half of the distinctly perceptible region would result in complaints from occupants. A floor that fell in the strongly perceptible range would be unacceptable to human occupants.

In accordance with Parmelee's (Murray 1975) recommendation that the modified Reiher-Meister scale be used instead of the Wiss-Parmelee rating system, Murray (1975) chose the former scale for determining vibration perceptibility. This decision resulted because the subjects of the Wiss-Parmelee research knew they were being tested, whereas the people involved in the modified Reiher-Meister research did not. The Wiss-Parmelee scale was therefore more severe than the modified Reiher-Meister scale.

Murray (1979) reported the results of a critical review of past floor vibration criteria and a description of a new criterion that greatly depended on damping. After testing 91 existing floors, he plotted the results (Figure 1.9) with the Murray (1975) criterion, the Wiss-Parmelee (1974) rating system, and a best fit line, corresponding to the following equation:

$$D = 35A_0f + 2.5 \tag{1.3}$$

in which D = percent of critical damping, A_0 = initial amplitude due to a single heel drop impact, in., and f = natural frequency, hz. The majority of plotted

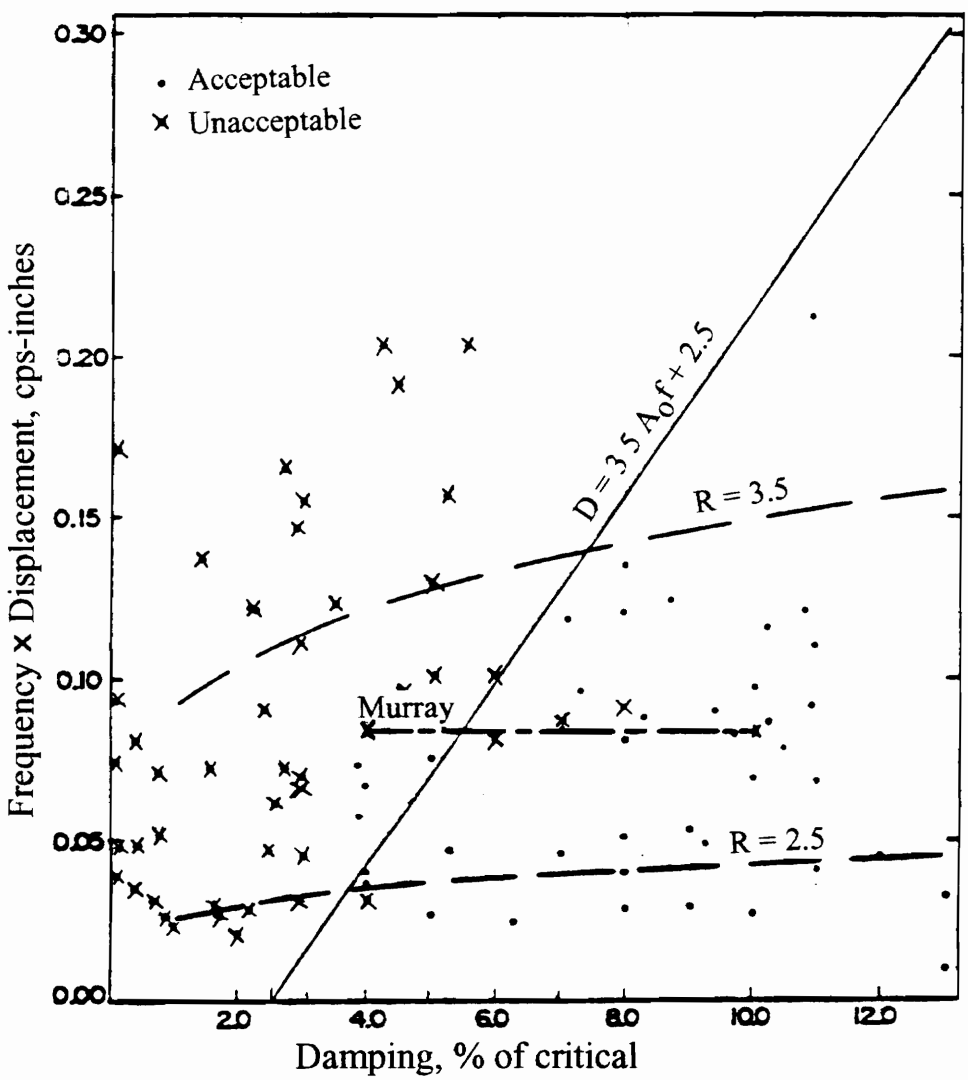


Figure 1.9 Comparison of Scale Ratings with Subjective Reactions [Murray 1979]

points to the left of the line represent unacceptable floors while most points to the right of the line represent acceptable floors.

As shown in Figure 1.9, the Wiss-Parmelee curves for R = 2.5 and R = 3.5 predict distinctly and strongly perceptible vibrations, respectively. A rating greater than 2.5 would indicate that redesign is necessary. From this graph, it can be concluded that the lower Wiss-Parmelee rating is conservative because it fails to adequately account for damping. This deemphasis on damping is explained by Wiss and Parmelee's study of human response to walking, in which vibration due to walking was nearly continuous at low damping levels.

Murray's (1975) criterion plotted in Figure 1.9 is valid for damping between 4 and 6% instead of 4 to 10% as suggested in the 1975 paper. It is conservative for damping values above 6% (Murray 1979).

The CSA criterion for 3, 6 and 12% damping are compared to Equation (1.3) for constant values of damping (Figure 1.10). The CSA criterion for 3% damping is unconservative for frequencies less than 7 hz in relation to Equation (1.3) for 3% damping. The additional disadvantage of the CSA criterion is that it only applies to three damping values, whereas Equation (1.3) applies to all damping values.

1.4 Introduction of a New Criterion

Allen (1991) proposes "a simple yet rational design criterion for walking vibration based on harmonic resonance". He places great emphasis on the fact that resonance can occur at the second and third harmonics of the step frequency

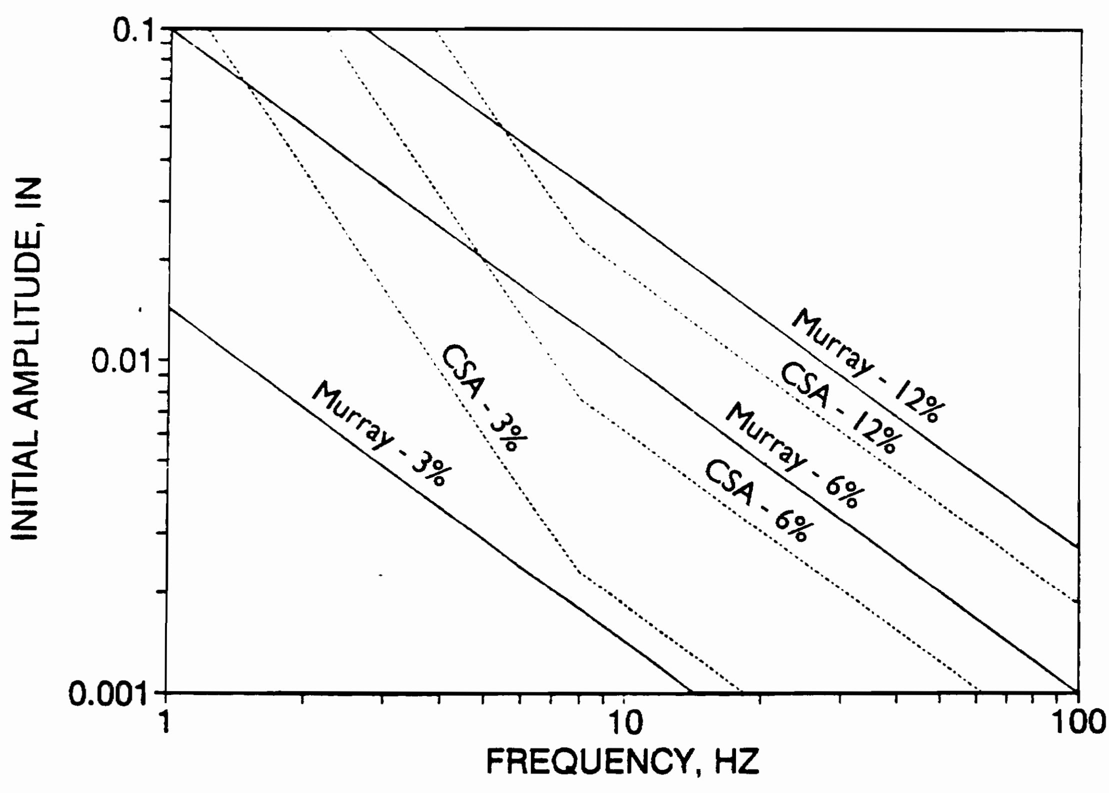


Figure 1.10 Comparison of Murray and CSA Criteria [Murray 1979]

for floors with natural frequencies of approximately 4 and 6 hz, respectively. Figure 1.11 shows test results for floors with natural frequencies that are first and second harmonics of the step frequency. For span 1, which has a fundamental frequency of 2.05 hz, the peak acceleration becomes increasingly large as the walking frequency approaches the span's frequency. In the case involving span 2, the peak acceleration increases sharply as the walking frequency nears 2.08 hz. This occurred because the natural frequency of span 2 is 4.17 hz, which is the second harmonic of the walking frequency. Based on these results, resonance should be considered when designing a floor for walking vibrations.

Allen accounts for resonance in his criterion by making floor acceptability largely dependent on the effective floor weight. As a floor natural frequency approaches the walking frequency, the required effective floor weight becomes larger to counter the effects of resonance. More energy is required to excite a heavier floor than a lighter one, thus reducing the floor's likelihood of achieving resonance due to walking impacts.

Further discussion of Allen's criterion is presented in the following chapters. In Chapter II, the criterion is derived and an explanation is given as to how the parameters are calculated. Chapter III describes the evaluation of the Allen criterion using the Murray criterion as a basis for comparison and the use of existing floors to determine the effectiveness of Allen's method. Finally, Chapter IV contains conclusions regarding the following investigation and recommendations for further research.

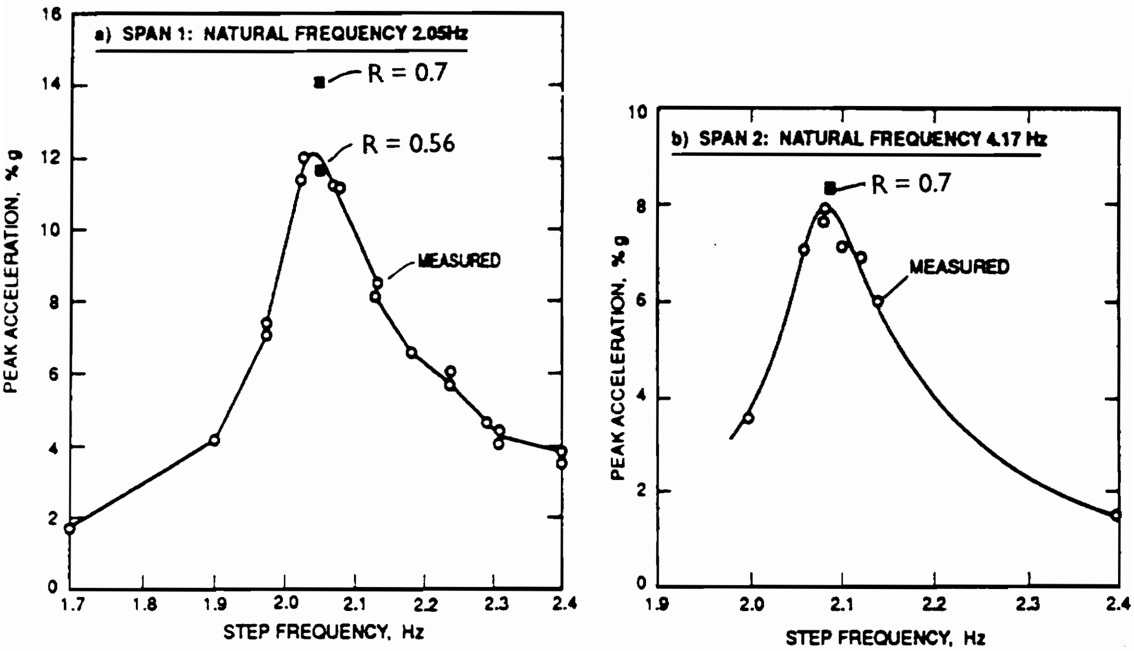


Figure 1.11 Peak Response of Two Footbridge Spans to a Person Walking Across at Different Step Frequencies [Allen 1991]

CHAPTER II

PRESENTATION OF ALLEN'S VIBRATION CRITERION

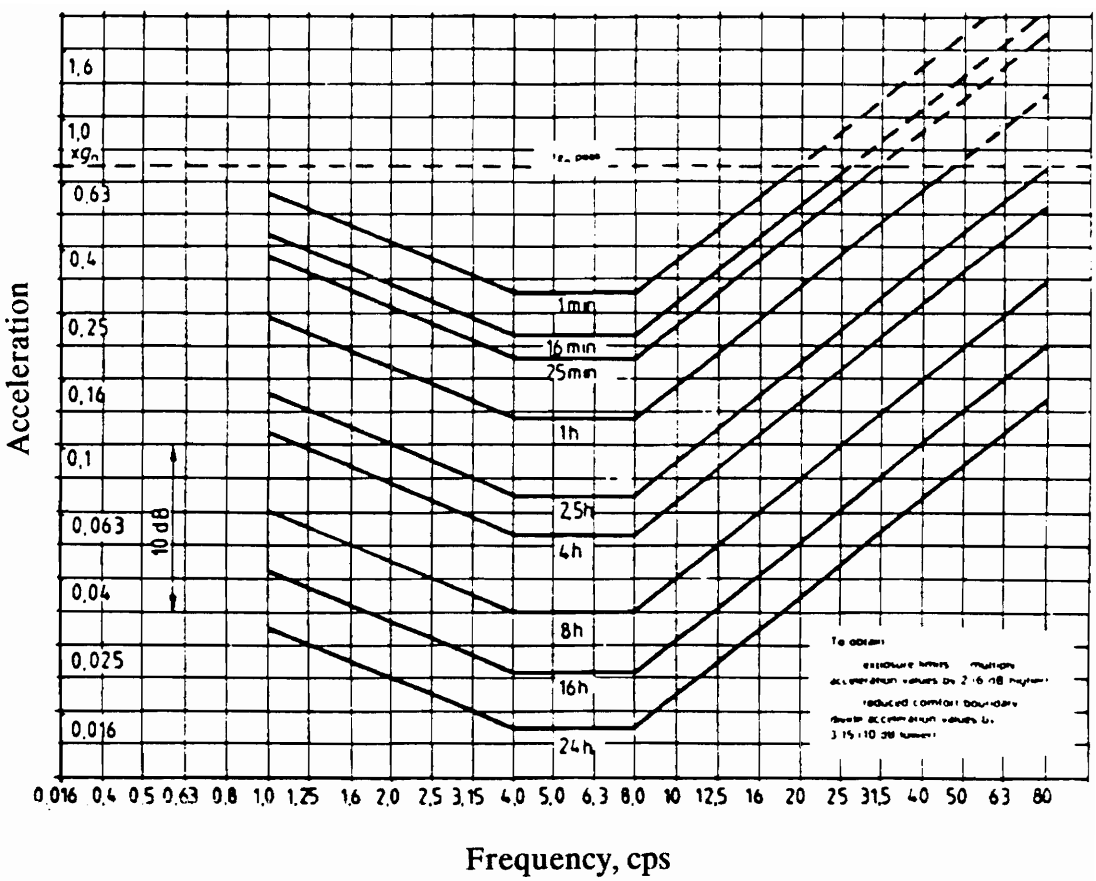
2.1 Acceleration Limits for Vibration

The International Standards Organization (ISO) suggests vibration limits for different building types that are deemed acceptable to occupants (Figure 2.1). Acceleration limits are given in terms of root mean square (rms) acceleration where:

$$a_{rms} = a_{peak} / 1.414 \qquad (2.1)$$

For offices, ISO (International Standards Organization 1985) recommends that the baseline curve, shown in Figure 2.1 as the four hour exposure curve, be multiplied by 4 for continuous or intermittent vibrations and suggests a multiplier of 128 for transient vibrations (Table 2.1). Intermittent vibrations consist of a discrete number of vibration incidents, resulting from a pile driver, for example. In contrast, transient vibrations occur more rarely. Walking is an intermittent vibration but it is less frequent and repetitive than a pile driver. As a result of this observation, Allen (1991) has multiplied the base curve by 10 for walking vibration in offices (Figure 2.2). In the frequency range of 4 to 8 hz, 0.005g is recommended as the acceleration limit based on the earlier work of Allen and Rainer (1976).

Due to the type of environment, footbridge and shopping mall occupants tend to be more tolerant of floor vibrations. Therefore, acceleration limits, shown



Longitudinal (a_Z) acceleration limits as a function of frequency and exposure time (fatigue-decreased proficiency boundary).

Figure 2.1 ISO Scale [ISO 1985]

TABLE 2.1 ADJUSTMENT FACTORS FOR ISO BASE CURVE
(ISO 1985)

Place	Time	Continuous or Intermittent Vibration	Transient Vibration
Critical Working Areas (e.g. operating rooms and precision labs)	Day	1	1
	Night	1	1
Residential	Day	2-4	60-90
	Night	1.4	20
Office	Day	4	128
	Night	4	128
Workshops	Day	8	128
	Night	8	128

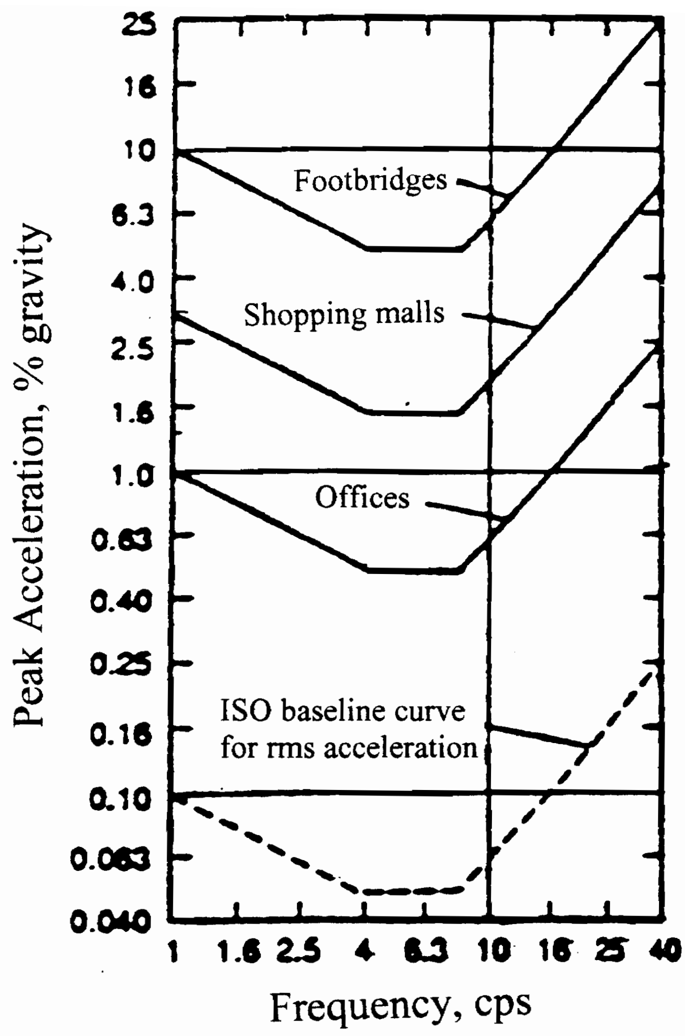


Figure 2.2 Recommended Acceleration Limits for Walking Vibration [Allen 1991]

in Figure 2.2, are less stringent for these types of structures with Allen (1991) recommending 0.014g and 0.049g for shopping malls and footbridges, respectively, in the frequency range of 4 to 8 hz.

2.2 Loading Function

Rainer, Pernica, and Allen (1988) measured dynamic forces during walking, running, and jumping using a platform 55.8 ft. long with load cells on a center support (Figure 2.3). At a rate indicated by prerecorded pulses playing through loudspeakers, test subjects walked across the platform. Figure 2.4 shows a typical force record at the middle support for one individual walking across the span. The record shows the dynamic forces, while excluding the static portion.

As shown in Figure 2.5, the Fourier, or frequency, spectrum shows that the second and third harmonics are present in addition to the step frequency. Because of the sinusoidal loading, the force, F, can be represented by the following equation (Allen 1991):

$$F = P(\,1 + \Sigma\, \alpha_i \cos(2\pi i f t)\,) \tag{2.2}$$

where P = weight of walker; f = step frequency, hz; i = harmonic multiple and α_i = Fourier amplitude, or coefficient. The dynamic portion of the loading function in Equation 2.2 is described by the summation term, which is a Fourier series with Fourier coefficients, α_i, at discrete frequencies, if (Rainer et al. 1988). The walking frequency, f, and α_i, are the most important terms describing the dynamic force. The Fourier coefficients, α_i, are called dynamic load factors, which are the ratio of dynamic force amplitude of each harmonic to the individual's weight. By

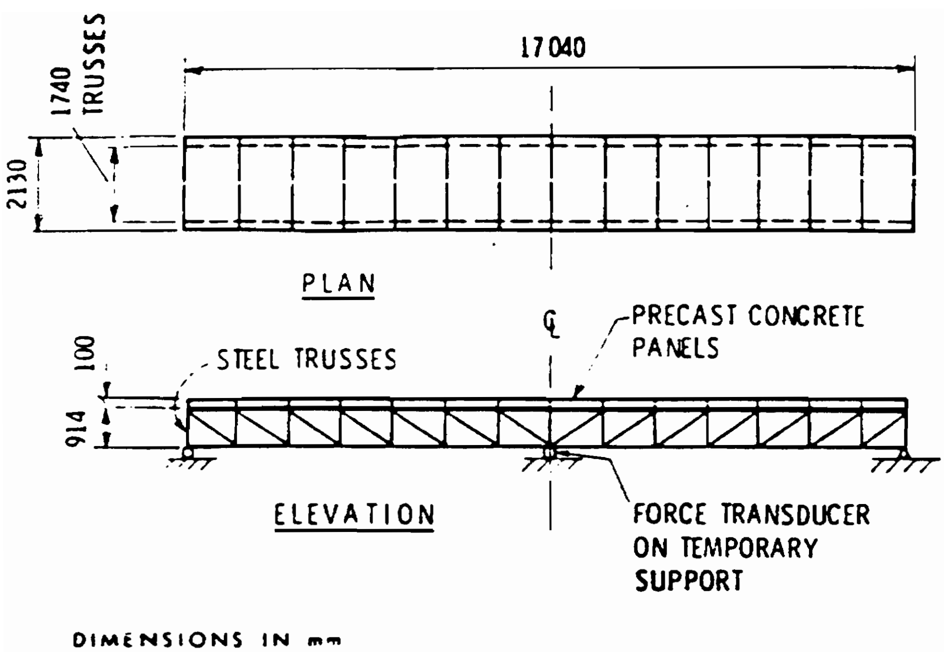


Figure 2.3 Instrumented Platform [Rainer et al 1988]

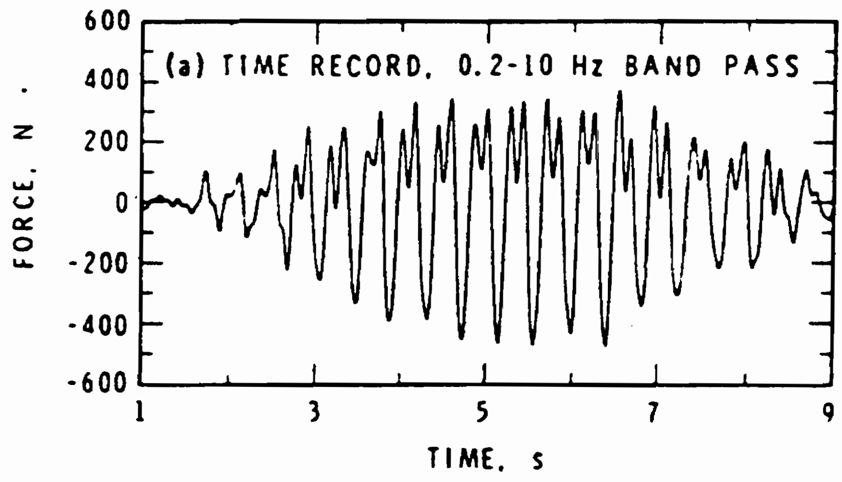


Figure 2.4 Force vs. Time Plot for Walking [Rainer et al 1988]

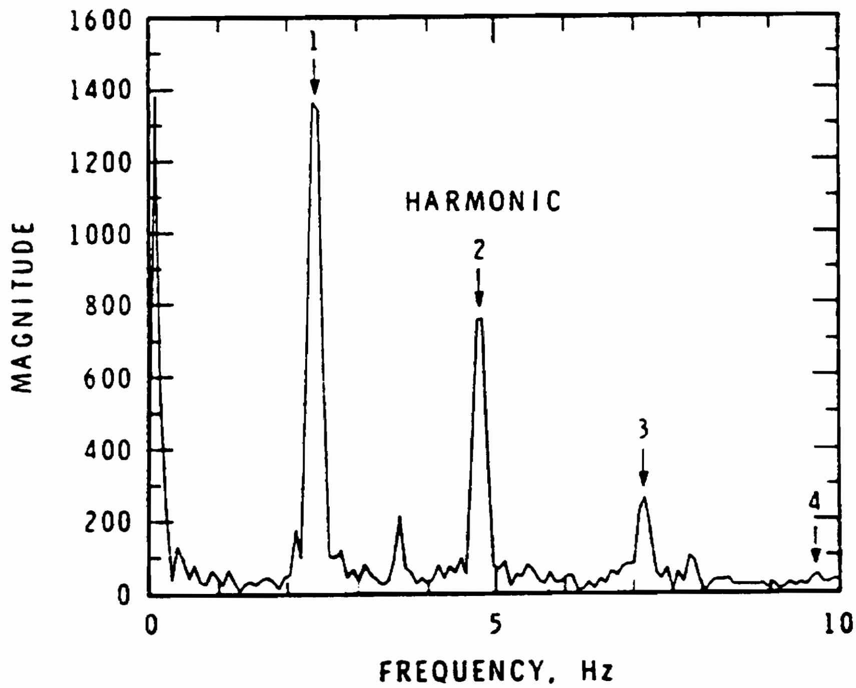


Figure 2.5 Fourier Amplitude Spectrum [Rainer et al 1988]

varying walking rates from 1.0 to 3.0 hz, a range of values for α_i was found (Figure 2.6). Since findings for individuals were similar to averaged results for the entire group, Rainer and Pernica (1986) presented the mean data.

2.3 Test Span Response to Walking

For dynamic evaluation, Rainer et al. (1988) used two simply supported test spans. After eliminating the middle support, span 1 was identical to the platform in Figure 2.3 and span 2 had the same length as span 1, but used two wide flange beams instead of trusses as the main structural members. Spans 1 and 2 had fundamental frequencies of 4.17 hz and 2.05 hz, respectively. An individual weighing 157 lbs. vibrated the spans at various walking frequencies, while span response was measured by vertical accelerometers at midspan.

For span 2, the peak acceleration at different walking rates is shown in Figure 2.7. The vertical axis of this plot is the ratio of peak acceleration to the subject's weight times the dynamic load factor for each walking rate (Rainer et al. 1988). When the walking rate corresponded to the fundamental frequency, the maximum acceleration occurred for the span, as expected. Rainer et al. (1988) compared the results to the steady-state response curve of the span shown by the dashed line in Figure 2.7. While the walking response at resonance is much lower than the steady-state results, the maximum response for walking frequencies above and below resonance is a little higher. At resonance, the variation can be explained by shorter duration and a changing force for walking (Figure 2.4). Above and below resonance, slightly larger walking accelerations were due to interference between the forcing function at one frequency and the excitation at

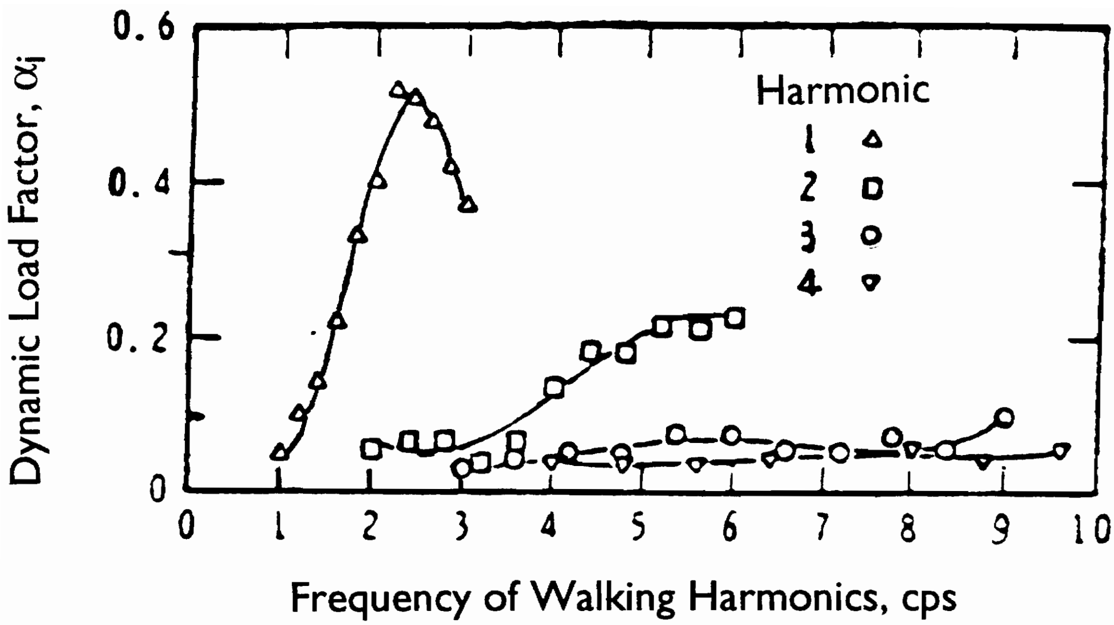


Figure 2.6 Averaged Dynamic Load Factors for Walking [Rainer et al 1988]

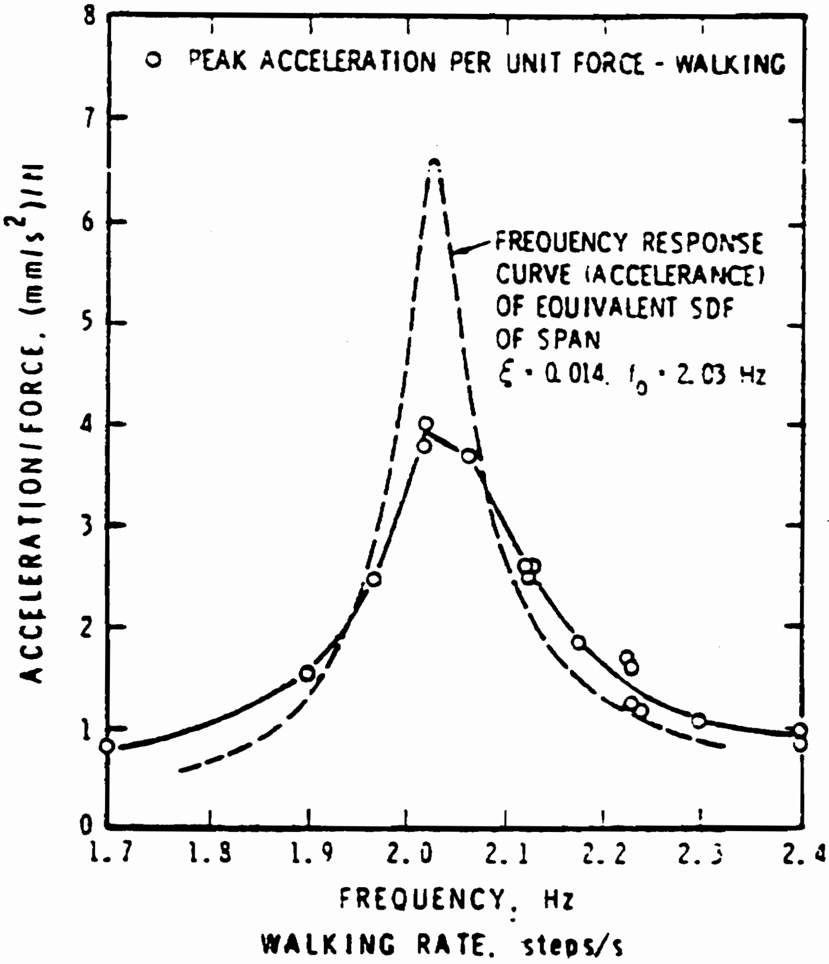


Figure 2.7 Peak Acceleration vs. Walking Rate [Rainer et al 1988]

the span natural frequency. Since maximum dynamic response occurred when the walking frequency coincided with the span natural frequency, this resulting resonance response is recognized as the worst vibration case and will be considered here in greater depth.

2.4 Calculation of Resonant Response to Walking

To calculate maximum response, the span is represented by a single degree of freedom oscillator. The equation for sinusoidal excitation at the walking frequency is given by

$$F = \alpha_i P \cos(2\pi i f t) \qquad (2.3)$$

Peak response is found at time $t = 0$ and Equation 2.3 reduces to (Rainer et al. 1988)

$$F = \alpha_i P \qquad (2.4)$$

As a further simplification to the problem, it will be assumed that steady-state response caused by harmonic resonance will occur even though it has been shown in Figure 2.7 that walking results in considerably smaller accelerations than steady-state motion. To account for the smaller response due to walking, the dynamic amplification factor caused by steady-state motion, G, is multiplied by a reduction factor, R. Equation 2.4 then becomes

$$F = \alpha_i P R G \qquad (2.5)$$

Besides accounting for a failure to achieve total steady-state resonance, the reduction factor, R, also recognizes that the person causing the vibration and the person sensing it are not at the same floor location of maximum amplitude, or acceleration. Based on the findings presented in Figure 1.11, Allen (1991) suggests that R should equal 0.7 for footbridges and 0.5 for floor structures exhibiting two-way action.

The dynamic amplification factor, G, for steady-state resonance can be found using the free body diagram shown in Figure 2.8. The differential equation of motion can be written as

$$ma + cv + kx = P_o \sin\omega t \tag{2.6}$$

where m = mass of single degree of freedom system; a = acceleration; v = velocity; x = displacement; P_o = static load; c = damping constant for the dashpot; k = spring constant and ω = frequency of the harmonic excitation. The above equation has a homogeneous solution and a particular solution. The homogeneous portion, however, results in damped free vibration, or transient vibration, and will only be present during the initial phases of motion and is therefore disregarded. On the other hand, the particular solution describes the steady-state harmonic oscillation at the frequency of the forcing function with the displacement vector lagging behind the force vector by a phase angle, Y. Therefore, the particular solution is (Thomson 1965)

$$x = X\sin(\omega t - Y) \tag{2.7}$$

where X is amplitude. By substituting into Equation 2.6, the following expression can be obtained (Thomson 1965):

$$m\omega^2 X\sin(\omega t - Y) - c\omega X\sin(\omega t - Y + \pi/2) - kX\sin(\omega t - Y) + P_o \sin\omega t = 0 \tag{2.8}$$

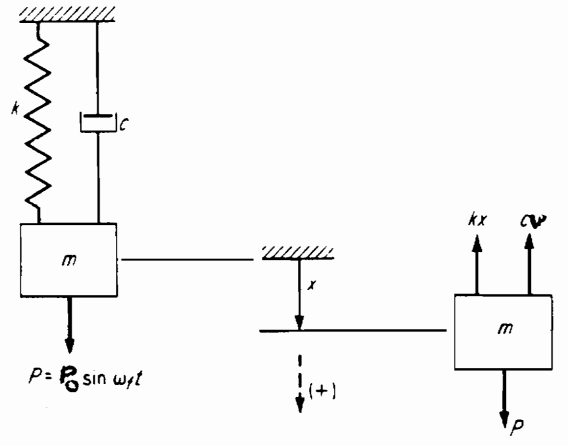


Figure 2.8 Free Body Diagram of Damped Single Degree of Freedom System with Harmonic Excitation [Vierck 1967]

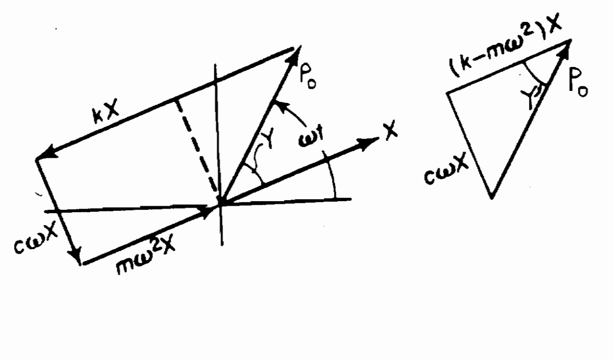


Figure 2.9 Vector Representation of Forced Vibration with Viscous Damping [Thompson 1965]

The above equation is shown in Figure 2.9 as a vector relation.

Using the Pythagorean Theorem and Figure 2.9, an expression can be found for X (Thomson 1965):

$$X = P_o/[(k-m\omega^2)^2+(c\omega)^2)]^{0.5} \tag{2.9}$$

After dividing the denominator and numerator by k, the equation becomes (Thomson 1965)

$$X = P_o/k/[(1-m\omega^2/k)^2+(c\omega/k)^2]^{0.5} \tag{2.10}$$

Since $\omega_n = (k/m)^{0.5}$ = natural frequency of undamped vibration in radians per second; D = damping ratio = c/c_c, then $c = c_cD$; $c_c = 2m\omega_n$ = critical damping coefficient; $X_o = P_o/k$ = displacement based on zero frequency for a spring-mass system acted on by a steady force P_o. Equation 2.9 can be expressed as (Thomson 1965)

$$G = X/X_o = 1/[(1-(\omega/\omega_n)^2)^2 + (2D(\omega/\omega_n)^2)^2]^{0.5} \tag{2.11}$$

For the case of steady-state resonance, $\omega = \omega_n$ and

$$G = 1/(2D) \tag{2.12}$$

where G = dynamic amplification factor.

2.5 Proposed Design Criterion

Recall that Equation 2.5 describes the force due to a single walking impact. This walking force can be written in terms of the resulting floor mass times the floor acceleration and Equation 2.5 becomes

$$ma = \alpha_i PRG \tag{2.13}$$

then dividing both sides by m gives:

$$a = \alpha_i PRG/m \tag{2.14}$$

and substituting $G = 1/(2D)$ gives:

$$a = \alpha_i PR/(2Dm) \tag{2.15}$$

where m = one-half of the total mass, M, of the effective floor for the equivalent single degree of freedom system modeling the span in Section 2.4. If the span had been mathematically represented as an infinite degree of freedom system, the total mass would have been used. By substituting $m = 0.5M$ and multiplying both sides of the equation by g, the acceleration due to gravity, the following equation is obtained (Allen 1991):

$$a/g = \alpha_i PR/(2D*0.5Mg) = R\alpha_i P/(DW) \tag{2.16}$$

Equation 2.16 shows that the peak acceleration response of a span is directly proportional to the dynamic load, $\alpha_i P$, and the amplification factor, $R/(2D)$, and inversely proportional to the total weight of the effective floor that participates in a vibration episode.

For design, Equation 2.16 can be expressed in terms of a minimum damping ratio, D, multiplied by the weight of the effective floor, W (Allen 1991):

$$DW > R\alpha_i P/(a/g) \tag{2.17}$$

When a floor is designed for an office or residential environment, R = 0.5, P = 157.4 lbs., a/g = 0.005 (Figure 2.2) and α_i = exp(-0.38f), based on Figure 2.6 where f is the floor natural frequency, hz. This results in the following equation:

$$DW > 15700\exp(-0.38f) \tag{2.17a}$$

Similarly, for shopping malls with R = 0.5, P = 157.4 lbs., a/g = 0.014 (Figure 2.2) and α_i = exp(-0.38f),

$$DW > 5620\exp(-0.38f) \tag{2.17b}$$

and for footbridges R = 0.7, P = 157.4 lbs., a/g = 0.049 (Figure 2.2) and α_i = exp(-0.38f),

$$DW > 2248\exp(-0.38f) \tag{2.17c}$$

More generally,

$$DW > K\exp(-0.38f) \tag{2.18}$$

which can be more conveniently expressed as

$$f > 2.63\ln(K/(DW)) \tag{2.19}$$

where K = 15700, 5620, and 2248 for offices, shopping malls, and footbridges, respectively. If the floor natural frequency, f, is greater than the required frequency defined by the terms to right of the inequality, then the floor or footbridge is acceptable to occupants for walking vibrations.

2.6 Damping Ratio, D

The amount of damping present in a floor system is largely dependent on the non-structural components, such as a hung ceiling, ductwork, partitions, and furniture. Table 2.2 presents Allen's (1991) recommended values for D and Table 2.3 displays Murray's (1991) suggested damping values. As can be seen from the two tables, Murray gives a broad range of damping values for different components, whereas Allen gives just one number for each structure type.

2.7 Natural Frequency, f

In determining natural frequency for a two-way floor system such as a steel beam- or steel joist-concrete slab arrangement, a dynamic modal analysis could be used. This process can be very difficult, however, when attempting to estimate the effect of non-structural components. Therefore, a simpler formula for predicting natural frequency will be used for the Allen criterion. The following equation which is derived by Timoshenko and Young text (1955) for prismatic bars,

$$f = 1.57(gE_sI_t/(wL^4))^{0.5} \tag{2.20}$$

where g = acceleration due to gravity, in/sec^2; E_s = modulus of elasticity for steel, psi; I_t = transformed moment of inertia of tee-section, in^4 (Figure 2.10); w = uniform weight of tee-section plus a fraction of the live load, lbs./in.; L = span, in.

TABLE 2.2 ALLEN'S RECOMMENDED VALUES FOR DAMPING, D
(Allen 1991)

Source	D
Footbridge	0.010
Bare Floor	0.015
Open Shopping Plaza	0.020
Finished Floor with Ceiling, Ducts, Flooring, and Furniture	0.030
Finished Floor with Partitions	0.045

TABLE 2.3 MURRAY'S SUGGESTED DAMPING VALUES (Murray 1991)

Source	D	Comments
Bare Floor	1%-3%	Lower limit for thin slab of lightweight concrete; upper limit for thick slab of regular weight concrete
Ceiling	1%-3%	Lower limit for hung ceiling; upper limit for sheetrock on furring attached to beams
Mechanical Systems	1%-10%	Depends on amount and attachment
Partitions	10%-20%	If attached to the floor system and not spaced more than every five floor beams or the effective joist floor width.

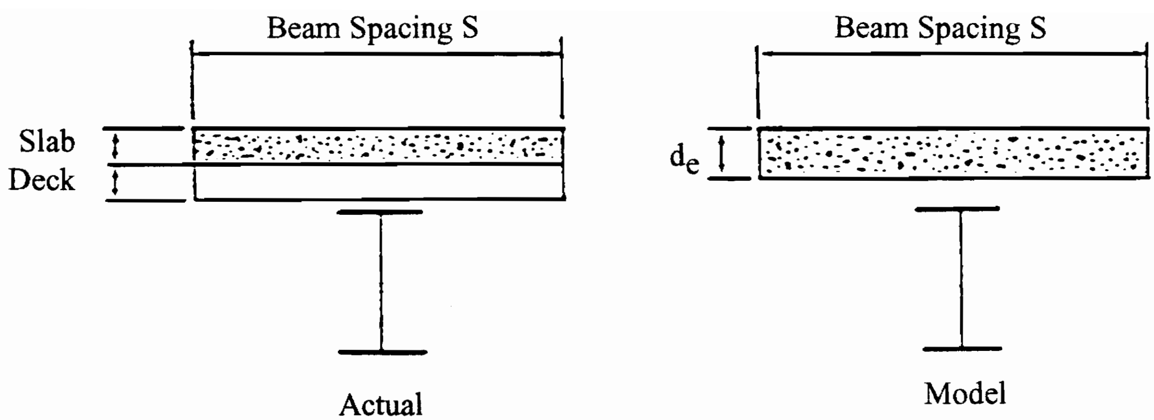


Figure 2.10 Tee-Beam Model for Computing Transformed Moment of Inertia [Murray 1991]

2.8 Joist Panel Mode

A joist panel mode occurs when a floor is impacted over the joists and, in response, the joists vibrate as a unit. The panel consists of the number of joists that participate in this vibration episode. For this mode, the girders supporting the joists make only a minor contribution to the vibration response, and the mode is therefore called a joist panel mode.

The natural frequency of the joist or beam can be calculated using Equation 2.20. Composite action is assumed between the concrete slab and a joist or beam even though shear connectors might not be present because small impact loadings caused by footfalls are not enough to overcome the friction at the deck/joist interface.

The weight of the joist panel can be found using (Allen 1991)

$$W_j = wB_jL_j \tag{2.21}$$

where w = dead weight of floor plus assumed live load, psf; L_j = joist span, ft.; B_j = width, ft., of the floor participating in a vibration episode which is determined from an equation based on orthotropic plate action (Allen 1991):

$$B_j = 2(D_s/D_j)^{0.25}L_j \tag{2.22}$$

where $D_s = E_c t^3/12$ and t = slab thickness, in., E_c = concrete modulus, psi; $D_j = E_s I_t/S$ where S = joist spacing, in. D_s is the flexural rigidity in the slab direction and D_j is the flexural rigidity in the joist direction based on composite action with the slab. The factor of 2 was determined by calibration to data found in Allen and Rainer's work (1976). Further, B_j must be limited to two-thirds of the length of the girder supporting the joists. If the joists are continuous over their supports and an adjacent span is $0.7L_j$ or greater, the joist panel weight, W_j, can be increased by one-half to account for a greater portion of the floor participating in a vibration episode (Allen 1991).

2.9 Girder Panel Mode

A girder panel mode consists of a single girder and the width of the effective floor that participates in a vibration episode. As in the case of the joist panel mode, the natural frequency of the girder panel can be predicted using Equation 2.20. The width of the girder panel can be approximated by (Allen 1991)

$$W_g = wB_gL_g \tag{2.23}$$

where L_g = girder span, ft. and B_g = width of effective floor, ft. determined from (Allen 1991)

$$B_g = 1.4(D_j/D_g)^{0.25}L_g \tag{2.24}$$

where $D_g = E_sI_t/S$ and I_t = girder transformed moment of inertia, in^4, S = girder spacing, in. D_j is the flexural rigidity in the joist direction and D_g is the flexural rigidity in the girder direction. The factor of 1.4 is used instead of 2 to account for the discontinuity of the joists at supports. The effective floor width, B_g, should be greater than the girder tributary width, but less than one and one-third times that width. If the girders are continuous over the supports and an adjacent span is $0.7L_g$ or more, the weight, W_g, can be increased by one-half (Allen 1991).

2.10 Combined Mode

Since joists and girders often vibrate as a unit rather than separately, it is important to consider the combined mode which results in a lower frequency due to the combined flexibility of the joists and girders. Dunkerly's equation can be used to estimate the system frequency when the joist has a lower frequency than the girder (Allen 1991):

$$1/f_s^2 = 1/f_g^2 + 1/f_j^2 \tag{2.25}$$

where f_s, f_g, and f_j are the system, girder and joist or beam frequencies, respectively. When the girder is more flexible, or has a lower frequency than the joist, the combined frequency should be taken as

$$f_s = \text{max.}(f_g \text{ or } f_j/1.414) \tag{2.26}$$

The combined weight can be estimated using the following equation:

$$W = W_j f_j/(f_j+f_g) + W_g f_g/(f_j+f_g) \tag{2.27}$$

2.11 Edge Mode

Interior, unsupported floor edges sometimes cause vibration problems because of reduced weight and decreased damping. Usually exterior edges are not a problem because they support walls and cladding. If an interior edge is supported by a joist, Allen (1991) indicates that the factor of 2 should be reduced to 1 in Equation 2.22. For an edge girder, he recommends that the weight be based on the tributary width (Allen 1991).

2.12 Natural Frequencies Greater than 10 hz

When the floor natural frequency is above 10 hz, resonance due to walking impacts is not a problem, but vibration complaints from occupants can still result (Allen 1991). Based on Allen's experience, floor deflection should be limited to 1mm (0.040 in.) for a 1 kN (225 lbs.) concentrated load as recommended by Ellingwood and Tallin (1984). For a joist panel mode, maximum deflection is calculated by placing the load at midspan of a joist. Then, the deflection is divided by the number of joists, found from the effective floor width, participating in a vibration episode. Maximum deflection for a girder panel mode is simply calculated by placing the 225 lb. concentrated load at midspan of the girder divided by the number of effective girders, which is one. For the combined mode, the total deflection is the sum of the joist deflection plus one-half the girder deflection. These limitations ensure adequate stiffness for office and residential buildings.

For lightweight floors with natural frequencies in the range of 10 to 20 hz, Allen (1991) indicates that annoying vibrations due to footstep impacts are caused by a lack of floor mass and inadequate damping. Equation 2.19 accounts for damping and floor weight and can, therefore, be applied to floors with natural frequencies greater than 10 hz. For floors above 17 hz, Allen has found that the 1kN/mm stiffness criterion begins to govern floor design. He recommends that both Equation 2.19 and the stiffness criterion should be applied to office and residential floors with natural frequencies greater than 10 hz in order to guarantee acceptable performance.

2.13 Conclusions

Using the allowable acceleration limits suggested by the ISO baseline scale, Allen established acceleration limits for offices, shopping malls, and footbridges. Then, a loading function was presented for walking with static and dynamic portions. The static portion was neglected since only the dynamic portion caused the vibration and is described by frequency and the dynamic load factor, determined from experimental results for walking harmonics. Through substitutions and rearranging of terms, the Allen criterion was derived for walking vibrations with emphasis placed on resonance. Next, it was shown how the floor natural frequency and effective floor weight could be found for the joist, girder, and combined modes. In conclusion, the Allen criterion is simple and straightforward to use with a firm basis in experimental results and a sound design objective: avoiding resonance with a floor due to walking.

CHAPTER III

EVALUATION OF THE PROPOSED ALLEN CRITERION

The Allen floor vibration criterion was presented in Chapter II. In this chapter, the proposed Allen criterion will be evaluated using the Murray criterion, Equation 1.3, as a basis. The Murray criterion was chosen as the reference because it has been widely accepted by the structural engineering profession in the United States.

3.1 Evaluation of Office Floors

Table 3.1 shows the results of vibration analyses of acceptable office floors using both the Murray and Allen criteria. (Floor properties and sample calculations for both criteria can be found in the appendix.) All of the floor systems presented in this table are located in existing office buildings and are acceptable to occupants with respect to walking vibrations.

For the Murray criterion, the estimated damping provided by the floor must be greater than the required damping for the floor to be acceptable. Damping provided by the floors shown in Table 3.1 is estimated at 4.5% based on values given in Table 2.3 for different floor components. Using this damping limit, all of the floor systems except those in Pittsburgh, Denver, and the DMW floors are acceptable. For example, the floor beam for the building in Denver is unacceptable because the required damping of 4.68% is greater than the

TABLE 3.1

ANALYSIS OF ACCEPTABLE OFFICE FLOORS

		f (hz)	A_0 (in.)	N_{eff}	$D_{req'd}$ (%)	B (ft.)	$F_{req'd}$ (hz)	Conclusion	
								Allen	Murray
Denver	Beam	8.55	.0073	1.88	4.68	26.7	6.90	A	U
	Girder	5.53	.0050	1.00	3.47	33.3	4.91	A	A
	System	4.64(6.05)	.0098	--	4.09	--	5.36	A	A
Pittsburgh	Beam	6.91	.0111	1.30	5.17	26.5	6.57	A	U
San Diego	Beam	6.68	.0086	1.53	4.51	29.1	6.00	A	A
	Girder	4.44	.0029	1.00	2.95	49.0	3.02	A	A
	System	3.70(4.72)	.0100	--	3.80	--	3.63	A	A
Stamford	Beam	6.65	.0077	1.84	4.30	20.0	7.17	U	A
	Girder	6.98	.0064	1.00	4.07	26.7	6.30	A	A
	System	4.82	.0109	--	4.34	--	6.72	U	A
Seattle	Beam	6.24	.0088	1.58	4.42	30.1	5.82	A	A
Denver	Beam	6.32	.0074	2.06	4.14	24.0	6.43	U	A
	Girder	6.26	.0040	1.00	3.39	39.9	4.73	A	A
	System	4.45(6.26)	.0094	--	3.97	--	5.44	A	A
Southfield, MI	Beam	4.30	.0076	2.30	3.64	20.0	6.37	U	A
	Girder	8.86	.0046	1.00	3.92	26.6	6.27	A	A
	System	3.87	.0099	--	3.84	--	6.35	U	A
Houston	Beam	5.27	.0076	1.90	3.91	23.3	6.08	U	A
	Girder	7.03	.0033	1.00	3.30	47.2	4.56	A	A
	System	4.22	.0093	--	3.87	--	5.43	U	A
Pittsburgh	Beam	5.45	.0081	1.46	4.05	29.5	5.03	A	A
Tampa	Beam	3.64	.0084	2.62	3.57	16.7	6.50	U	A
	Girder	7.16	.0067	1.00	4.17	29.9	6.50	A	A
	System	3.25	.0118	--	3.84	--	6.50	U	A

TABLE 3.1 (Continued)

ANALYSIS OF ACCEPTABLE OFFICE FLOORS

		f (hz)	A_0 (in.)	N_{eff}	$D_{req'd}$ (%)	B (ft.)	$F_{req'd}$ (hz)	Conclusion Allen	Murray
San Diego	Beam	4.11	.0065	2.00	3.43	33.4	4.08	A	A
	Girder	4.09	.0026	1.00	2.87	66.7	2.24	A	A
	System	2.90(4.09)	.0077	--	3.28	--	3.00	A	A
DMW 1422	Beam	9.61	.0130	1.57	6.87	21.8	8.72	A	U
DMW 1631	Beam	9.55	.0104	2.15	5.97	21.5	8.10	A	U
DMW 1626	Beam	9.45	.0111	2.45	6.16	21.6	8.08	A	U
DMW 813	Beam	16.71	.0106	1.88	8.69	16.6	10.82	A	U
1 a	Joist	5.35	.0052	2.52	3.48	16.7	6.27	U	A
	Girder	10.14	.0020	1.00	3.21	64.0	5.25	A	A
	System	4.73	.0062	--	3.53	--	6.01	U	A

estimated damping of 4.5%. For the Denver and DMW floors, the beam frequencies are above 8 hz which fall in the range where Murray's criterion becomes overly conservative. It should also be noted that the system frequency for Murray's criterion is calculated using Equation 2.25 from Chapter II. Unlike Allen's criterion for the combined mode, this equation is used even though the beam or joist frequency may be greater than the girder frequency.

The Allen criterion is also used to evaluate the floor systems in Table 3.1. Recall that the natural frequency of the floor must be greater than the required frequency for the floor to be acceptable. Unlike the Murray criterion, the Allen standard correctly predicts that the Denver floor and the DMW floors are acceptable. However, the Allen criterion incorrectly predicts the performance of other office floors having frequencies lower than 8 hz. As a beam or girder frequency approaches one of the three walking harmonics, the required frequency increases and the required effective floor weight participating in a vibration episode increases. This trend towards increased floor weight and required frequency to avoid resonance due to walking causes the Allen criterion to be overly conservative for floors below 8 hz.

For the Allen criterion, when the beam frequency is greater than the girder frequency, the system frequency is taken as the girder frequency, or the beam frequency divided by 1.414, depending on which value is greater. This value is indicated in parentheses in Table 3.1 and any tables that follow. When the girder frequency is greater than the beam frequency, Equation 2.25 is used to calculate the system, or combined, frequency.

Table 3.2 summarizes the vibration analyses of existing office floors that have received vibration complaints from occupants and are therefore unacceptable. The Murray and Allen criteria do equally well in predicting floor performance. For girders or beams (joists) with frequencies below 8 hz in Table 3.2, the Allen criterion is more likely to indicate that the floor is unaccceptable for walking vibrations. This is in accordance with the criterion goal of avoiding floor resonance with the three walking harmonics. Moreover, the effective floor width, B, is often limited in Table 3.2, and the following tables, by the limits placed on girder and joist panel width. The analysis results using the Murray criterion do not indicate any general trends, as they did for Table 3.1.

A summary of the evaluation results for the acceptable and unacceptable office floors using the Murray and Allen criteria is presented in Table 3.3. For this tabulation, floor systems are considered to be above 8 hz if the beam and/or girder have a frequency greater than this limit. If neither the girder nor the beam has a frequency above 8 hz, then a floor is a "less than 8 hz" floor. Further, a floor system is unacceptable if the beam (joist) , girder, or system receives an unacceptable rating. In other words, only one of the three floor components must be found unacceptable for the entire floor system to be considered unacceptable. As indicated in Table 3.3, the Allen criterion more accurately predicts floor performance above 8 hz with a 77% success rate versus a 46% prediction rate for the Murray criterion. The Murray criterion, however, correctly predicts the vibration performance for 77% of the floors below 8 hz as opposed to a 54 % success rate for the Allen criterion.

TABLE 3.2

ANALYSIS OF UNACCEPTABLE OFFICE FLOORS

		f (hz)	A_0 (in.)	N_{eff}	$D_{req'd}$ (%)	B (ft.)	$F_{req'd}$ (hz)	Conclusion Allen	Murray
Dubuque Iowa (352)	Joist	8.00	.0103	6.55	5.39	13.2	8.73	U	U
	Girder	15.00	.0049	1.00	5.09	22.8	8.60	A	U
	System	7.06	.0128	--	5.65	--	8.70	U	U
Star (139)	Beam	8.46	.0061	1.67	4.32	26.7	6.39	A	A
	Girder	4.29	.0056	1.00	3.43	31.9	4.45	U	A
	System	3.82(5.97)	.0090	--	3.70	--	4.75	A	A
Temple or Shalon (349)	Joist	9.86	.0086	6.00	5.48	15.3	9.03	A	U
	Girder	12.82	.0035	1.00	4.06	27.7	7.26	A	A
	System	7.82	.0104	--	5.34	--	8.23	U	U
Evergreen Int'l Aviation (340)	Joist	5.82	.0100	3.25	4.54	27.1	5.71	A	U
Ford Aero Bldg. (315)	Joist	7.12	.0072	2.40	4.30	14.0	7.19	U	A
	Girder	9.40	.0030	1.00	3.49	44.0	6.14	A	A
	System	5.68	.0087	--	4.23	--	6.76	U	A
AT&T Omaha (436)	Joist	7.72	.0169	3.27	7.08	13.3	9.02	U	U
	Girder	10.25	.0051	1.00	4.34	32.3	7.68	A	A
	System	6.15	.0195	--	6.71	--	8.45	U	U
Firehouse (422)	Joist	9.14	.0191	2.51	8.60	19.9	9.08	A	U
	Girder	6.00	.0069	1.00	3.96	40.1	4.66	A	A
	System	5.01(6.46)	.0225	--	6.45	--	5.40	A	U
New York Church (412)	Beam	6.87	.0065	2.05	4.06	26.8	5.74	A	A
	Girder	7.23	.0033	1.00	3.33	23.9	6.62	A	A
	System	4.98	.0081	--	3.91	--	6.12	U	A
Harrisburg, PA (392)	Joist	6.16	.0069	7.13	3.98	26.3	5.81	A	A
DMW 1835	Beam	7.25	.0119	1.81	5.51	25.1	7.45	U	U

TABLE 3.3

RESULTS OF OFFICE FLOOR EVALUATIONS

	Prediction Rate for Floors					
	Above 8hz		Below 8hz		Combined	
	Allen	Murray	Allen	Murray	Allen	Murray
Acceptable Floors	71%(5/7)	29%(2/7)	56%(5/9)	89%(8/9)	63%(10/16)	63%(10/16)
Unacceptable Floors	83%(5/6)	67%(4/6)	50%(2/4)	50%(2/4)	70%(7/10)	60%(6/10)
Combined	77% (10/13)	46%(6/13)	54%(7/13)	77%(10/13)	65%(17/26)	62%(16/26)

3.2 Evaluation of Unacceptable Shopping Malls

In Table 3.4, results are shown for the analysis of shopping malls using the Ellingwood and Tallin (1986) criterion in addition to the Murray and Allen criteria. For the Ellingwood and Tallin criterion, a floor or footbridge is acceptable if the static deflection caused by a 450 pound concentrated load, divided by the number of effective tee-beams and placed anywhere on the floor, is less than 0.02 in. (See the appendix for sample calculations of the Ellingwood and Tallin criterion.)

Because Murray (1991) recommends the use of the Ellingwood and Tallin criterion in lieu of his own for shopping malls, it is included in this portion of the study. Murray makes this recommendation because his criterion is meant to be used for transient vibrations, where damping plays an important role in determining the acceptability of a floor. While an office floor is more likely to experience transient vibrations caused by one or two people walking across the floor, a shopping mall floor is subject to groups of people continuously walking across the floor. This constant movement can cause a vibration that is nearly steady-state and damping therefore becomes less critical. Instead of damping, floor stiffness becomes the most important factor in determining floor acceptability. For this reason, Murray recommends the stiffness criterion presented by Ellingwood and Tallin.

As shown in Table 3.4, however, none of the three criteria do well in predicting vibration performance for shopping mall floors. Neither the Allen criterion nor the Ellingwood and Tallin criterion correctly predicts the

TABLE 3.4

ANALYSIS OF UNACCEPTABLE SHOPPING MALLS

		f (hz)	A_0 (in.)	N_{eff}	$D_{req'd}$ (%)	B (ft.)	$F_{req'd}$ (hz)	Δ/N_{eff} (in.)	Conclusions: Allen	Conclusions: Murray	Conclusions: Ellingwood & Tallin
4a	Beam	7.02	.0071	2.31	4.23	26.7	4.23	.0056	A	A	A
	Girder	6.32	.0058	1.00	3.78	20.0	4.07	.005	A	A	A
	System	4.70(6.32)	.0099	--	4.14	--	4.14	.0081	A	A	A
4c	Beam	7.02	.0071	2.31	4.23	13.3	6.07	.0056	A	A	A
	Girder	9.19	.0038	1.00	3.71	36.7	4.45	.0025	A	A	A
	System	5.58	.0089	--	4.24	--	5.35	.0069	A	A	A
4e	Beam	5.93	.0059	2.44	3.73	26.7	3.38	.0054	A	A	A
	Girder	5.41	.0031	1.00	3.08	46.6	1.91	.003	A	A	A
	System	3.99(5.41)	.0075	--	3.55	--	2.48	.0069	A	A	A
5a	Joist	7.76	.0145	3.18	6.43	18.7	6.00	.0107	A	U	A
	Girder	7.53	.0046	1.00	3.70	37.2	4.03	.0034	A	A	A
	System	5.40(7.53)	.0167	--	5.67	--	4.81	.0124	A	U	A

unacceptability of any of the floors. Of the four floor systems, the Murray criterion correctly predicts the performance of one.

3.3 Evaluation of Pedestrian Bridges

The analysis of pedestrian bridges using the Allen, Murray, and Ellingwood and Tallin criteria is shown in Table 3.5. In actuality, four of the five footbridges are unacceptable, while the fifth specimen vibrates in a barely perceptible manner due to walking and is therefore judged to be acceptable. The Allen criterion correctly predicts the floor performance for all of the bridges and the Murray criterion correctly predicts the acceptability/unacceptability of four out of five of the structures. Murray (1991) recommends using his criterion for pedestrian bridges that are lightly travelled, and thus subject to transient vibrations rather than continuous vibration caused by a greater number of people. Floor 2a might be heavily travelled, which would explain why the Murray criterion incorrectly predicts the floor's acceptability. The Ellingwood and Tallin criterion correctly predicts floor performance for three of the five footbridges.

3.4 Conclusions

Based on evaluation of existing floors using the Allen, Murray and Ellingwood and Tallin vibration criteria, it can be concluded which is most effective for the offices, shopping malls, and pedestrian bridges found in this study. The Murray criterion best predicted the floor performance for offices with girder and beam frequencies below 8 hz. For office floors with a girder or beam frequency above 8 hz, the Allen criterion was more accurate than

TABLE 3.5

ANALYSIS OF PEDESTRIAN BRIDGES

		f (hz)	A_0 (in.)	N_{eff}	$D_{req'd}$ (%)	B (ft.)	$F_{req'd}$ (hz)	Δ/N_{eff} (in.)	Conclusions: Allen	Conclusions: Murray	Conclusions: Ellingwood & Tallin	Subj. Eval.
DOUGFLR	Joist	7.30	.0288	2.00	9.86	6.7	8.92	.0222	U	U	U	U
LABFLR	Joist	7.64	.0683	2.00	20.77	4.0	13.59	.0513	U	U	U	U
2a	Beam	4.68	.0112	2.00	4.33	10.5	5.04	.0123	U	A	A	U
	Girder	4.06	.0044	1.00	3.12	5.3	2.24	.0055	A	A	A	U
	System	3.07(4.06)	.0134	--	3.93	--	3.11	.0151	A	A	A	U
3a	Beam	7.07	.0135	2.00	5.85	8.0	6.83	.0107	A	U	A	U
	Girder	24.73	.0243	1.00	23.56	1.5	13.44	.0114	A	U	A	U
	System	6.80	.0257	--	8.62	--	7.02	.0164	U	U	A	U
6a	Beam	4.79	.0070	2.00	3.67	10.0	3.60	.0151	A	A	A	A

the Murray criterion. In the case of shopping malls, none of the three criteria satisfactorily predicted floor performance, while the Allen criterion did the best in predicting the acceptability/unaccepptability of footbridges.

CHAPTER IV

CONCLUSIONS AND RECOMMENDATIONS

4.1 Conclusions

Lightweight floor systems are susceptible to annoying vibrations caused by walking. Since improvement of the vibration characteristics for an existing floor is costly and impractical, it is best to design a floor that is free of vibration problems. With this design goal in mind, existing criteria were examined. Of these, the Murray criterion was found to work best for transient vibrations because it adequately accounted for damping, while the others did not. The Murray criterion was calibrated for floors in the 5-8 hz range and it becomes inaccurate beyond 8 hz.

The Allen criterion was then presented as a solution to the inaccuracy found in the Murray criterion. Unlike the Murray criterion which places great importance on the role of damping, the Allen criterion emphasizes the avoidance of walking resonance with a floor. As a floor frequency nears the walking frequency (approximately 2 hz) effective floor weight required by the Allen criterion becomes exponentially large.

To determine which criteria is better, existing office floors were evaluated using the Murray and Allen methods. The Murray criterion worked best for floors under 8 hz, and the Allen criterion more accurately predicted floor performance above 8 hz.

In addition to the Allen and Murray criteria, the Ellingwood and Tallin criterion was used to evaluate shopping malls and pedestrian bridges. This static deflection limitation was included because it emphasizes the importance of stiffness for floors subject to nearly continuous vibration from walking. The Ellingwood and Tallin criterion, however, proved to be unconservative for shopping malls and footbridges. The Allen and Murray criteria were also inaccurate for shopping malls, but both criteria did well in predicting the acceptability of footbridges.

4.2 Recommendations

Based on the evaluation results for the floor data given in Chapter III, it is suggested that the Allen and Murray criteria be used in a complementary fashion. For an office environment, the Murray criterion should be applied to floors below 8 hz while the Allen criterion should be used to check floors above 8 hz. Based on the results, neither criterion should be applied to shopping malls, and both can be used to evaluate pedestrian bridges. Recall, however, that the Murray criterion should only be applied to footbridges that are lightly travelled, such as in an hotel atrium (Murray 1991).

In the case of shopping malls and pedestrian bridges, the amount of data available was scarce and further investigation into these structures might result in findings contrary to the results presented in this thesis. Therefore, it is recommended that the data base for pedestrian bridges and shopping malls be expanded so that the constants used in the Allen criterion can be adjusted to account for this more extensive data. The Allen criterion would then be more accurate when predicting the acceptability of these strcture types.

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APPENDIX

TABLE A.1

PROPERTIES OF ACCEPTABLE OFFICE FLOORS

		Size	L (ft.)	LL (psf)	SPA (in.)	I_4 (in.)	b_{eff} (in.)	t_c (in.)	γ_c (pcf)	f_c' (psi)	A (in.^{2})	BM. C.G. to Top of Slab (in.)	DL (psf)
Denver	Beam	W14x22	25.0	11	108	199	108	4.75	110	3500	6.49	13.12	46
	Girder	W27x84	40.0	14	300	2850	108	4.75	110	3500	24.80	19.61	47
Pittsburgh	Beam	W16x26	28.3	11	120	301	120	3.50	150	3500	7.68	12.35	46
San Diego	Beam	W16x26	29.0	11	120	301	120	4.00	150	3500	7.68	13.35	53
	Girder	W36x160	50.0	14	480	9750	100	4.00	150	3500	47.00	23.50	54
Stamford	Beam	W16x26	30.0	11	120	301	120	4.75	110	3500	7.68	14.10	46
	Girder	W21x44	30.0	14	240	843	120	4.75	110	3500	13.00	16.58	46
Seattle	Beam	W16x26	30.0	11	120	301	120	4.00	150	3500 3500	7.68	13.35	53
Denver	Beam	W16x31	32.7	11	108	375	108	4.75	110	3500	9.12	14.19	47
	Girder	W27x84	36.0	14	360	2850	108	4.75	110	3500	24.80	19.60	46
Southfield, MI	Beam	W16x36	40.0	11	120	448	120	4.75	110	3500	10.60	14.18	47
	Girder	W24x62	30.0	14	240	1500	120	4.75	110	3500	18.20	18.12	47
Houston	Beam	W21x44	41.0	11	120	843	120	4.25	110	3500	13.00	15.58	43
	Girder	W27x114	35.0	14	426	4090	120	4.25	110	3500	33.50	18.90	42
Pittsburgh	Beam	W24x55	43.5	11	120	1350	120	3.50	150	3500	16.20	1630	49
Tampa	Beam	W18x35	45.0	11	100	510	100	3.50	150	3500	10.30	13.35	48
	Girder	W18x35	25.0	14	270	510	100	3.50	150	3500	10.30	13.35	45
San Diego	Beam	W24x55	50.0	11	120	1350	120	4.00	150	3500	16.20	17.30	56
	Girder	W36x160	50.0	14	600	9750	120	4.00	150	3500	47.00	23.50	53

TABLE A.1 (Continued)

PROPERTIES OF ACCEPTABLE OFFICE FLOORS

		Size	L (ft.)	LL (psf)	SPA (in.)	I_4 (in.)	b_{eff} (in.)	t_c (in.)	γ_c (pcf)	f_c' (psi)	A (in.^{2})	BM. C.G. to Top of Slab (in.)	DL (psf)
DMW 1422	Beam	W14x22	25.0	0	100	199	100	3.37	115	2820	6.49	10.87	35
DMW 1631	Beam	W16x31	29.9	0	72	375	72	3.37	115	2820	9.12	11.94	37
DMW 1626	Beam	W16x26	29.9	0	60	301	60	3.37	115	2820	7.68	11.85	38
DMW 813	Beam	W8x13	15.0	0	72	39.6	72	3.37	115	2820	3.84	8.00	34
	Joist	40LH875	62.3	0	100	6995	100	4.75	110	3500	7.12	26.25	46
1a	Girder	W24x62	25.0	0	742	1550	100	4.75	110	3500	18.20	23.25	45

TABLE A.2

PROPERTIES OF UNACCEPTABLE OFFICE FLOORS

		Size	L (ft.)	LL (psf)	SPA (in.)	Depth (in.)	Top of Joist to Top of Slab (in.)	I_4 (in.)	b_{eff} (in.)	t_c (in.)	γ_c (pcf)	f_c' (psi)	A (in.^{2})	BM. C.G. to Top of Slab (in.)	DL (psf)
Dubuque Iowa (352)	Joist	24H07	34.3	11	24	24	2.5	239	24	2.22	150	4000	1.80	--	30
	Girder	W18x40	19.8	14	206	--	--	612	46	2.22	150	4000	11.80	13.93	30
Star (139)	Joist	Builtup	24.0	11	120	--	--	244	120	4.75	150	3000	5.15	14.50	61
	Girder	Builtup	40.0	14	288	--	--	1788	120	4.75	150	3000	16.00	18.75	61
Temple or Shalon (349)	Beam	16H5	23.0	5	30	16	3.6	86	30	3.28	150	3500	1.53	--	43
	Girder	W18x50	23.0	8	276	--	--	1313	57	3.28	150	3500	20.70	15.56	44
Evergreen Int'l Aviation (340)	Joist	24LH08	40.1	11	48	--	--	524	48	3.25	150	3000	3.92	16.00	44
Ford Aero Bldg. (315)	Joist	36LH534	44.0	11	64	36	4.0	1741	64	3.25	150	3500	5.68	--	44
	Girder	31LH99	21.0	14	528	--	--	822	64	3.25	150	3500	3.85	24.50	41
AT&T Omaha (436)	Joist	18H8	30.0	11	34	18	2.5	185	34	2.28	150	3500	2.56	--	32
	Girder	W16x40	20.0	14	360	--	--	518	45	2.28	150	3500	11.80	13.00	30
Firehouse (422)	Joist	12K3	18.0	11	48	12	3.5	36	48	2.75	150	3000	1.20	--	35
	Girder	W24x104	41.0	14	216	--	--	3100	56	2.75	150	3000	30.60	18.00	40
New York Church (412)	Beam	W18x65	38.0	11	96	--	--	1070	96	4.25	110	3000	19.10	14.43	47
	Girder	W27x94	32.0	14	456	--	--	3270	96	4.25	110	3000	27.70	18.71	41
Harrisburg, PA (392)	Joist	26K-J1	40.0	11	30	25	3.6	414	30	3.30	145	3000	2.90	--	44
DMW 1835	Beam	W18x35	35.0	0	100	--	--	510	100	3.37	110	3224	10.30	12.85	35

TABLE A.3

PROPERTIES OF UNACCEPTABLE SHOPPING MALLS

		Size	L (ft.)	LL (psf)	SPA (in.)	Depth (in.)	Top of Joist to Top of Slab (in.)	I_4 (in.)	b_{eff} (in.)	t_c (in.)	γ_c (pcf)	f_c' (psi)	A (in.^{2})	BM. C.G. to Top of Slab (in.)	DL (psf)
4a	Beam	W16x26	30.0	10	80	--	--	301	80	4.50	150	3000	7.68	13.35	60
	Girder	W27x84	40.0	13	180	--	--	2850	80	4.50	150	3000	24.80	18.86	62
4c	Beam	W16x26	30.0	10	80	--	--	301	80	4.50	150	3000	7.68	13.35	60
	Girder	W18x35	20.0	13	360	--	--	510	80	4.50	150	3000	10.30	14.35	57
4e	Beam	W21x44	40.0	10	80	--	--	843	80	4.50	150	3000	13.00	15.83	63
	Girder	W30x124	40.0	13	420	--	--	5360	80	4.50	150	3000	36.50	20.59	60
5a	Joist	22H6	28.0	27	34	22	3.5	171	34	2.84	110	3000	1.50	--	28
	Girder	W24x62	28.0	30	336	--	--	1550	61	2.84	110	3000	18.20	17.87	28

TABLE A.4

PROPERTIES OF PEDESTRIAN BRIDGES

		Size	L (ft.)	LL (psf)	SPA (in.)	Depth (in.)	Top of Joist to Top of Slab (in.)	I_4 (in.)	b_{eff} (in.)	t_c (in.)	γ_c (pcf)	f_c' (psi)	A (in.^{2})	BM. C.G. to Top of Slab (in.)	DL (psf)
DOUGFLR	Joist	12	30.0	0	40	--	--	86	40	2.75	150	4000	3.25	10.96	38
LABFLR	Joist	14K1	24.7	2	24	14	2.5	37.6	24	2.22	145	3000	--	--	28
2a	Beam	W24x55	50.0	5	63	--	--	1350	63	4.75	120	3500	16.20	14.35	58
	Girder	W24x55	30.3	545	65	--	--	1350	63	4.75	120	3500	16.20	14.35	58
3a	Beam	W21x50	33.5	3	48	--	--	984	0	3.75	150	3500	14.70	0	59
	Girder	W10x22	13.4	5	17	--	--	118	0	3.75	150	3500	6.49	0	62
6a	Beam	W21x147	56.0	10	54	--	--	3630	54	3.25	150	3000	43.20	15.03	73

TABLE A.5
DYNAMIC LOAD FACTORS FOR HEEL-DROP IMPACT [Murray 1991]

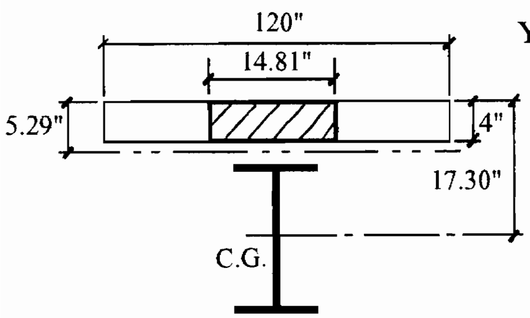
f, Hz	DLF	F, Hz	DLF	F, Hz	DLF
1.00	0.1541	5.50	0.7819	10.00	1.1770
1.10	0.1695	5.60	0.7937	10.10	1.1831
1.20	0.1847	5.70	0.8053	10.20	1.1891
1.30	0.2000	5.80	0.8168	10.30	1.1949
1.40	0.2152	5.90	0.8282	10.40	1.2007
1.50	0.2304	6.00	0.8394	10.50	1.2065
1.60	0.2456	6.10	0.8505	10.60	1.2121
1.70	0.2607	6.20	0.8615	10.70	1.2177
1.80	0.2758	6.30	0.8723	10.80	1.2231
1.90	0.2908	6.40	0.8830	10.90	1.2285
2.00	0.3058	6.50	0.8936	11.00	1.2339
2.10	0.3207	6.60	0.9040	11.10	1.2391
2.20	0.3356	6.70	0.9143	11.20	1.2443
2.30	0.3504	6.80	0.9244	11.30	1.2494
2.40	0.3651	6.90	0.9344	11.40	1.2545
2.50	0.3798	7.00	0.9443	11.50	1.2594
2.60	0.3945	7.10	0.9540	11.60	1.2643
2.70	0.4091	7.20	0.9635	11.70	1.2692
2.80	0.4236	7.30	0.9729	11.80	1.2740
2.90	0.4380	7.40	0.9821	11.90	1.2787
3.00	0.4524	7.50	0.9912	12.00	1.2834
3.10	0.4667	7.60	1.0002	12.10	1.2879
3.20	0.4809	7.70	1.0090	12.20	1.2925
3.30	0.4950	7.80	1.0176	12.30	1.2970
3.40	0.5091	7.90	1.0261	12.40	1.3014
3.50	0.5231	8.00	1.0345	12.50	1.3058
3.60	0.5369	8.10	1.0428	12.60	1.3101
3.70	0.5507	8.20	1.0509	12.70	1.3143
3.80	0.5645	8.30	1.0588	12.80	1.3185
3.90	0.5781	8.40	1.0667	12.90	1.3227
4.00	0.5916	8.50	1.0744	13.00	1.3268
4.10	0.6050	8.60	1.0820	13.10	1.3308
4.20	0.6184	8.70	1.0895	13.20	1.3348
4.30	0.6316	8.80	1.0969	13.30	1.3388
4.40	0.6448	8.90	1.1041	13.40	1.3427
4.50	0.6578	9.00	1.1113	13.50	1.3466
4.60	0.6707	9.10	1.1183	13.60	1.3504
4.70	0.6835	9.20	1.1252	13.70	1.3541
4.80	0.6962	9.30	1.1321	13.80	1.3579
4.90	0.7088	9.40	1.1388	13.90	1.3615
5.00	0.7213	9.50	1.1434	14.00	1.3652
5.01	0.7337	9.60	1.1519	14.10	1.3688
5.20	0.7459	9.70	1.1583	14.20	1.3723
5.30	0.7580	9.80	1.1647	14.30	1.3758
5.40	0.7700	9.90	1.1709	14.40	1.3793

SAMPLE CALCLULATIONS USING THE MURRAY CRITERION

Check the following floor system for susceptibility to vibration:

2-1/2" normal weight (150 pcf, n = 8.1) concrete slab
3" metal deck; assume live load of 11psf.
- 50'-0" long W24x55's frame into each side of a 50'-0" long W36x160. The beams are spaced at 10'-0" o.c.

Beam W24x55 A = 16.20 in.^{2} I_x = 1350in.^{4} Bm. c.g. to top of slab = 17.30 in.



$$Y_b = \frac{(14.81x4.0)(4.0/2)+16.20(17.30)}{75.44} = 5.29\text{in.}$$

$$I_t = \frac{14.81(4.0)^3}{12} + 59.24(5.29-4.0/2)^2$$

$$+1350 + 16.20(17.30-5.29)^2$$

$$= 4407\text{in.}^4$$

Supported weight = Slab + Live Load (11psf) + Beam

W = ((4.0/12)(150)(10) + ((11)(10) + 55)50 = 33,250 lbs.

$$f_b = 1.57\left[\frac{gEI_t}{WL^3}\right]^{0.5} = 1.57\left[\frac{(386.0)(29x10^6)(4407)}{(33,250)(50x12)^3}\right]^{0.5} = 4.11\text{hz}$$

From Table A.5, DLF = 0.6063

$$A_{ot} = (DLF)_{max}\frac{600L^3}{48EI_t} = \frac{(0.6063)(600)(600)^3}{(48)(29x10^6)(4407)} = 0.01281\text{in.}$$

$$N_{eff} = 2.97 - \frac{S}{17.3d_e} + \frac{L^4}{1.35EI_t}$$

$$= 2.97 - \frac{120}{(17.3)(4.0)} + \frac{(600)^4}{(1.35)(29x10^6)(4407)} = 1.987$$

$A_{ob} = A_{ot}/N_{eff}$ = 0.01281/1.987 = 0.0065 in.

Required Damping = 35 $A_o f$ + 2.5 = 35(0.0065)4.11 + 2.5 = 3.43%

Girder W36x160 A = 47.00 in^2 I_x = 9750 in^4 Bm. c.g. to top of slab = 23.50 in.

I_t = 21,955 in^4, using an effective slab width of 10 ft.

W = ((4.0/12)(150) + 16.5)(50x50) + 160(50) = 174,250 lbs.

$$f_g = 1.57\left[\frac{(386.0)(29x10^6)(21955)}{(174250)(600)^3}\right]^{0.5} = 4.01hz$$

$(DLF)_{max}$ = 0.5929

$$A_{ot} = \frac{(0.5929)(600)(600)^3}{(48)(29x10^6)21955} = 0.0025 1in.$$

N_{eff} = 1.0 A_{og} = 0.0025 in.

$D_{req'd}$ = 35(0.0025)4.01 + 2.5 = 2.85%

System

$$\frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{4.11^2} + \frac{1}{4.01^2}$$

f_s = 2.87 hz

$A_{os} = A_{ob} + A_{og} / 2$ = 0.0065 + 0.0025 / 2 = 0.0078 in.
$D_{req'd}$ = 35(0.0078)2.87 + 2.5 = 3.28%

Evaluation

Estimated Damping = Bare Floor (3%) + Ceiling, Ductwork, Mechanical (1.5%) = 4.5%

	f, hz	A_o, in.	$D_{req'd}$, %	
Beam	4.11	0.0065	3.43 < 4.5	Acceptable
Girder	4.01	0.0025	2.85 < 4.5	Acceptable
System	2.87	0.0078	3.28 < 4.5	Acceptable

Conclusion: Floor is acceptable for supporting office and residential environments.

SAMPLE CALCULATIONS FOR THE ALLEN CRITERION

Check the same interior bay used in the "SAMPLE CALCULATIONS FOR THE MURRAY CRITERION".

Beam Panel Mode

f_b = 4.11 hz (same as before)

$E_c = 33(w)^{1.5}(f'c)^{0.5} = 33(150)^{1.5}(3500)^{0.5} = 3.587\text{x}10^6$ psi

$$D_s = \frac{E_c t^3}{12} = \frac{(3.587\text{x}10^6)(4)^3}{12} = 19.131\text{x}10^6 \text{ in.}-\text{lbs}$$

$$D_b = \frac{E_s I_t}{S} = \frac{(29\text{x}10^6)(4407)}{(10\text{x}12)} = 1.065\text{x}10^9 \text{ in.}-\text{lbs.}$$

$$B_b = 2\left[\frac{D_s}{D_b}\right]^{0.25} \text{x}L = 2\left[\frac{19.131\text{x}10^6}{1.065\text{x}10^9}\right]^{0.25} \text{x}50 = 36.6\text{ft.}$$

Panel Width = Girder Length = 50 ft.

$B_b = 36.6 > .667(50) = 33.4$ ft., Use $B_b = 33.4$ ft.

W = 33250 / 10 x 33.4 = 111,055 lbs.

$$F_{req'd} = 2.63\ln\left[\frac{K}{DW}\right] = 2.63\ln\left[\frac{15,700}{(0.03)(111,055)}\right] = 4.08\text{hz}$$

Girder Panel Mode

f_g = 4.01 hz (same as before)

$D_b = 1.065\text{x}10^9$ in.-lbs.

$$D_g = \frac{E_s I_t}{S} = \frac{(29\text{x}10^6)(21,955)}{(50\text{x}12)} = 1.061\text{x}10^9 \text{ in.}-\text{lbs.}$$

$$B_g = 1.4\left[\frac{D_b}{D_g}\right]^{0.25} \text{x}L_g = 1.4\left[\frac{1.065\text{x}10^9}{1.061\text{x}10^9}\right]^{0.25} \text{x}50 = 70.1\text{ft.}$$

Panel Width = Girder Tributary Width = 50 ft.

$B_g = 70.1 > 50$ ft.

$> 1.33(50) = 66.7$ ft., Use $B_g = 66.7$ ft.

W = 174,250 / 50 x 66.7 = 232,450 lbs.

$$F_{req'd} = 2.63\ln\left[\frac{15,700}{(0.03)(232,450)}\right] = 2.13\text{hz}$$

Combined Mode

$$\text{Since } f_b > f_g,\ f_s = \max.\left\{\frac{f_g = 4.01\text{hz}}{f_b / 1.414 = 4.11\text{hz} / 1.414 = 2.91\text{hz}}\right\}$$

$f_s = f_g = 4.01$ hz

$$W = \frac{W_b f_b}{(f_b + f_g)} + \frac{W_g f_g}{(f_b + f_g)}$$

$$= \frac{(111,055)(4.11)}{(4.11 + 4.01)} + \frac{(232,450)(4.01)}{(4.11 + 4.01)} = 171,005\text{lbs.}$$

$$F_{req'd} = 2.63\ln\left[\frac{15,700}{(0.03)(171,005)}\right] = 2.94\text{hz}$$

EVALUATION

	f, hz	B, ft.	$F_{req'd}$, hz	
Beam Panel	4.11	33.4	4.08	Acceptable
Girder Panel	4.01	66.7	2.13	Acceptable
Combined Mode	4.01	--	2.94	Acceptable

Conclusion: Floor vibration due to walking will not annoy occupants.

SAMPLE CALCULATIONS FOR THE ELLINGWOOD AND TALLIN CRITERION

Check the interior bay used in the previous two examples.

Beam

$$\Delta_b = \frac{PL^3}{48EI_t} = \frac{(450)(600)^3}{(48)(29x10^6)(4407)} = 0.0158\text{in.}$$

$N_{eff} = 1.987$ (From the example using the Murray criterion)

$\Delta_b / N_{eff} = 0.0158 / 1.987 = 0.0080$ in. < 0.02 in., Acceptable

Girder

$$\Delta_g = \frac{PL^3}{48EI_t} = \frac{(450)(600)^3}{(48)(29x10^6)(21,955)} = 0.0032\text{in.}$$

$N_{eff} = 1.00$

$\Delta_g / N_{eff} = 0.0032$ in. < 0.02 in., Acceptable

System

$\Delta_s = \Delta_b + \Delta_g / 2 = 0.0158 + 0.0032 / 2 = 0.0174$ in. < 0.02 in., Acceptable

Conclusion: Floor is acceptable for walking vibrations.

VITA

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