

FLOOR VIBRATIONS: GIRDER EFFECTIVE MOMENT OF INERTIA AND COST STUDY

Chapter 1

Introduction and Literature Review

1.1 Introduction

Annoying floor vibrations have become a major serviceability problem in recent years. The prevalence of the problem has grown because the use of high-strength steel and lightweight concrete has led to buildings with longer member spans and less stiffness, mass, and damping than with previous designs. These factors tend to result in floors that are more susceptible to problematic vibrations.

Occupant induced floor vibrations pose no threat to structural integrity. Rather, they can cause varying degrees of discomfort. Extreme cases render buildings unusable and the required corrections are expensive. However, problems can be avoided with sufficient design. The general goal of this research is to improve upon the existing design guidelines for preventing annoying vibrations of joist supported floor systems.

The research consists of two separate studies: 1) the effective moment of inertia of girders that support concrete slabs using joist seats as the horizontal shear connections, and 2) a cost efficiency analysis comparing composite and non-composite floor systems that meet vibration design standards. Both studies have applications in floor design. The effective moment of inertia study may be used to improve the accuracy in predicting the behavior of joist supported systems. The cost study may give a designer a starting point for designing cost effective floors that meet vibration criteria.

Research on girder effective moment of inertia is the subject of Chapter 2. The current method of predicting girder effective moment of inertia does not account for some new trends in floor design. These trends include the use of larger and/or compositely constructed joists, usually with wide spacing. These joists typically have taller joist seats than do the joists used at narrower spacing. Two typical examples of recent problematic floors had bays of 40 ft x 40 ft and 28 ft x 40 ft. For both floors, LH-series joists at the girder quarter points

were used to span the 40 ft direction, providing only one shear connector in each half span. The floors were probably more flexible than predicted causing them to vibrate at a lower frequency. The tall seats and wide spacing of the joists used in these floors are likely causes for the problems.

Chapter 2 contains results from tests on eight 30 ft x 8 ft “footbridges” with girders or joist-girders spanning the 30 ft dimension and joists transverse to the girders. Joist spacing and joist type were varied between the floors. For seven footbridges, bolted, welded, and reinforced seat-to-girder connections were tested, providing three complete sets of testing for each footbridge. The order of testing was reinforced (as constructed), bolted, then welded. The eighth footbridge was a special case in that tubing was added between each pair of joist seats in an attempt to attain full composite stiffness. Using results from static, dynamic, and seat stiffness testing and FE modeling, assessment of the current method of predicting girder effective moment of inertia was made. Improvements on the method of prediction are proposed.

The study on the cost of composite and non-composite floors is the subject of Chapter 3. Recently, composite construction has been used to improve cost efficiency by reducing structural weight and in some cases by reducing story height. However, vibration problems are a design consideration in composite floors because lighter floors tend to be more lively. To determine which type construction is more economical when vibration criteria are met, four typical size bays were considered: 28 ft x 40 ft, 30 ft x 40 ft, 30 ft x 30 ft, and 40 ft x 40 ft. Using the design software “SDI FLOOR” (Elhouar and Murray 1993), all acceptable configurations of member sizes and spacing were evaluated for composite and non-composite cost. Three separate trials were conducted, with initial dead load deflection limits and unit stud cost as the variables.

The remainder of Chapter 1 contains terminology relevant to the text, a review of vibrations literature including recently conducted research, and a “need for research” section.

1.2 Terminology

The terms defined below will be used often throughout the text. The terms are common in any textbook on structural vibrations, but are tailored here to the particular area of floor vibrations.

Loading. *Static loads* are those loads that remain constant over time. Static loading can be used to ascertain floor stiffness. *Dynamic loads* are loads that are a function of time. The load induced by walking is an example of *transient* dynamic loading. A heel drop impact is an example of *impulsive* dynamic loading. Bouncing is an example of *periodic* dynamic loading. *Walking loads* are simply the loads induced by walking footfalls. Walking excitation is the primary cause of the problematic floor vibrations under consideration. Heel drop excitation is commonly used for testing purposes. A standard *heel drop* is the 2.5 in. toes-to-heels drop of a 170 lb person. Figure 1.1 shows the “standard” heel drop loading function (Murray 1975).

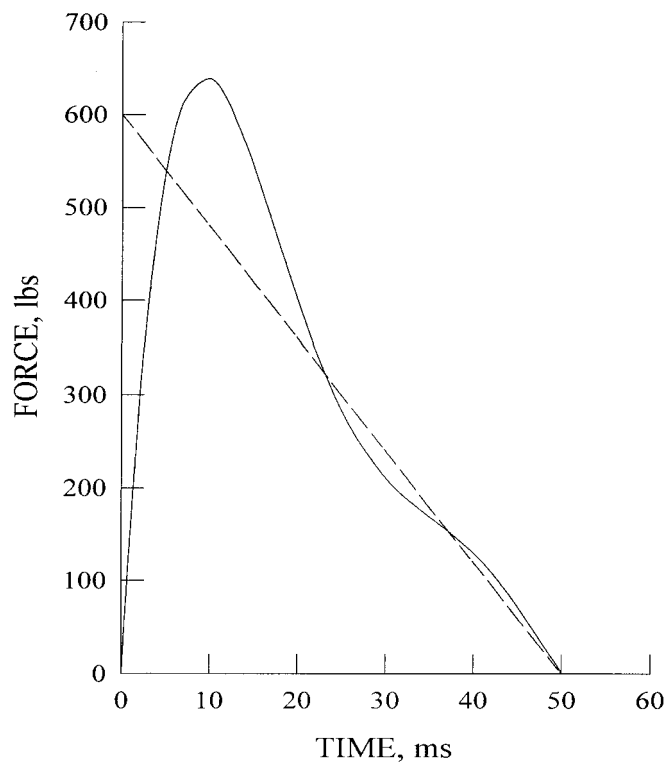


Figure 1. 1—Heel Drop Loading

Vibration. *Floor vibration* is the movement of the floor above and below its position of equilibrium. *Excitations* are any actions that cause vibration. *Free vibration* occurs after a system has been excited and is left to vibrate under its own spring-like action. *Forced vibration* results from continuous excitation.

Amplitude, Damping, and Frequency. *Amplitude* is magnitude of motion expressed in terms of displacement, velocity, or acceleration. The vertical axis of Figure 1.2 shows the

amplitude of a typical sine wave. Amplitude will often be expressed as a fraction of gravitation acceleration. *Period* is the time between *cycles* or successive instances of both equal magnitude and sense of motion. *Frequency* is the number of cycles that occur per unit time (also the inverse of period). *Hertz (Hz)* is the unit for frequency used in this text (cycles per second). *Natural frequencies* are frequencies that have significant amplitude during free vibration. The *fundamental natural frequency* is the lowest of the natural frequencies. Unless otherwise indicated, “natural frequency” can be understood to be the fundamental natural frequency. A *frequency spectrum* shows the relative contribution of frequencies. Figure 1.3 is an example frequency spectrum.

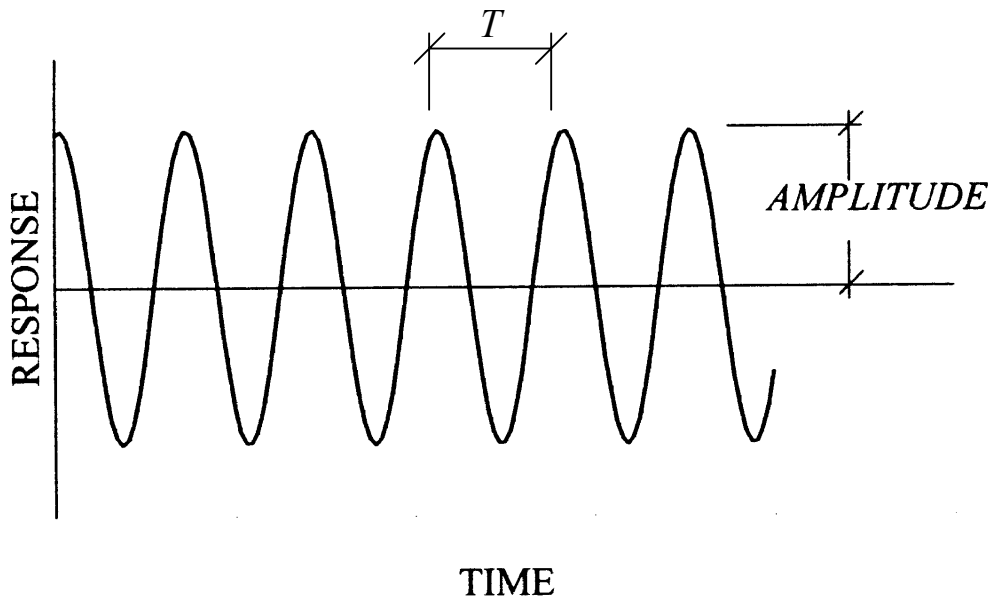


Figure 1. 2—Sine Wave

Resonance occurs when excitation is applied at or near a natural frequency or multiple of natural frequency. Resonance results in large vibrating amplitudes relative to magnitude of excitation. Conversely, excitation at non-resonant frequencies periodically resists the motion of the floor. *Damping* is the ability of the system to dissipate energy. This can be anything, including people, that dissipates energy. Damping reduces and eventually eliminates the amplitude of a vibration.

Effective Moment of Inertia. The term *effective moment of inertia*, or I_{eff} , is often used to define the behavior of sections whose flexural stiffness results from two or more connected members that resist bending together.

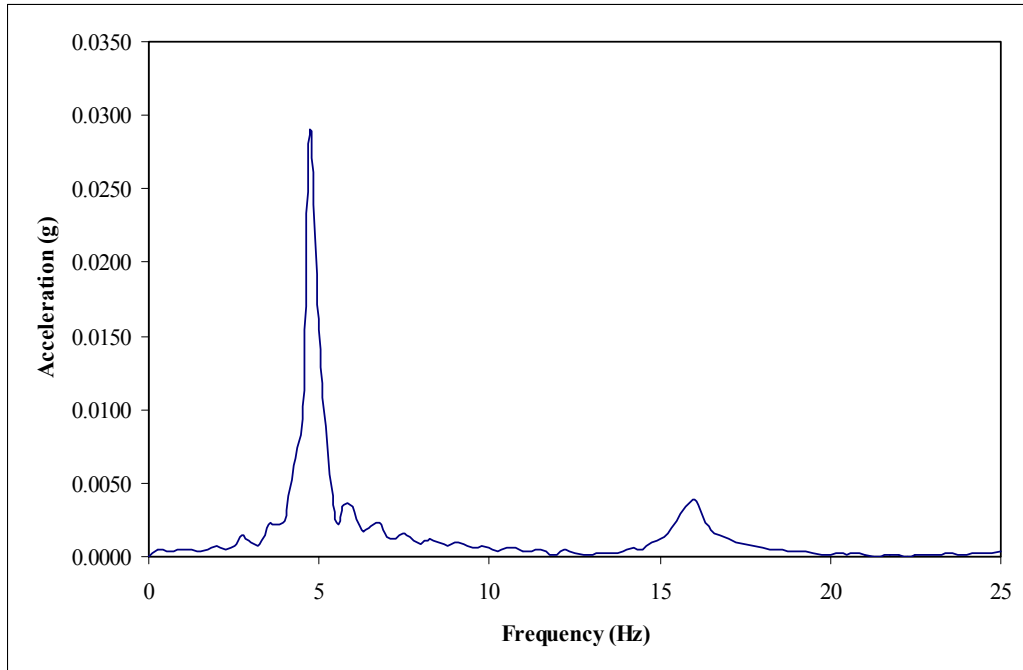


Figure 1. 3—Frequency Spectrum

1.3 Literature Review

The major factors that affect vibration behavior are stiffness, mass, and damping; each is expensive to increase. Structural engineers would not have problems preventing problematic vibrations, if cost were not of concern. Of course, cost efficiency is important. Nevertheless, correcting a vibration problem can be much more expensive than the extra cost that may be required in prevention by design. Hence, not unlike many other aspects of structural engineering, the extensive body of work that exists on the subject of floor vibrations has been driven by the struggle to balance efficiency and function.

Historical floor vibrations design criteria upon which much of the current criteria are based are discussed in the following section. The currently used criteria are discussed in Section 1.3.2. Some criteria that do not directly relate to this research are included to help put the topics covered in this research in context. Research not yet reflected in the criteria is covered in Section 1.3.3.

1.3.1 Historical Floor Vibrations Criteria

Seven decades ago, engineers began creating design criteria for the floor vibration problem. In 1931, Reiher and Meister wrote the first such criteria (Lenzen 1966). The Rei-

her–Meister scale predicts perception of vibration as a function of displacement and frequency. To produce the scale, ten men subjected to steady state (cycles of constant magnitude) vibration, categorized their perception as either “not perceptible”, “slightly perceptible”, “distinctly perceptible”, or “strongly perceptible”. Testing included frequencies of 5-70 Hz and amplitudes of 0.001-0.04 in.

Lenzen (1966) reported that “variations in frequency and amplitude within the range measured in the field had a minor effect” on test subjects’ perception. Instead, damping had the greater effect. This was because damping can significantly reduce the magnitude of a vibration as it propagates through a floor. According to Lenzen, when damping reduced vibrations to negligible amplitudes before five oscillations (complete cycles of motion), subjects only felt the initial impact. Perception increased when larger amplitude remained after five oscillations. However, only above 12 oscillations was perception as severe as with steady state motion. Lenzen concluded that typical floor systems had enough damping to justify increasing the displacements deemed acceptable by the Reiher-Meister scale by a factor of ten. This result is reflected in the Modified Reiher-Meister scale, shown in Figure 1.4. Leading to an additional modification, Murray (1975) found that “ steel beam, concrete slab systems, with relatively open areas free of partitions and damping between 4 and 10 percent, which plot above the upper one-half of the distinctly perceptible range, will result in complaints from the occupants”. The resulting modification is labeled “R-M/Murray Criterion” on the Modified Reiher-Meister scale of Figure 1.4.

In his 1966 paper, Lenzen also suggested guidelines for approximating stiffness, frequency, and displacement. He reported that with special considerations, the joist full composite moment of inertia, I_t , could be used in predicting frequency. I_t is taken as the sum of effective joist T-sections. For a T-section, the sum of half of the distance to each neighboring joist gives the effective width of concrete. When designing bays with more than ten joists, he recommended assuming the sum of only ten T-sections. Lenzen noted that the basis of these stiffness recommendations was on floors with 2.5 in. slabs and 2 ft joist spacing. He found close approximation of system natural frequency, f , with

$$f = 1.57 \sqrt{\frac{gEI_t}{wdl^4}} \quad (1.1)$$

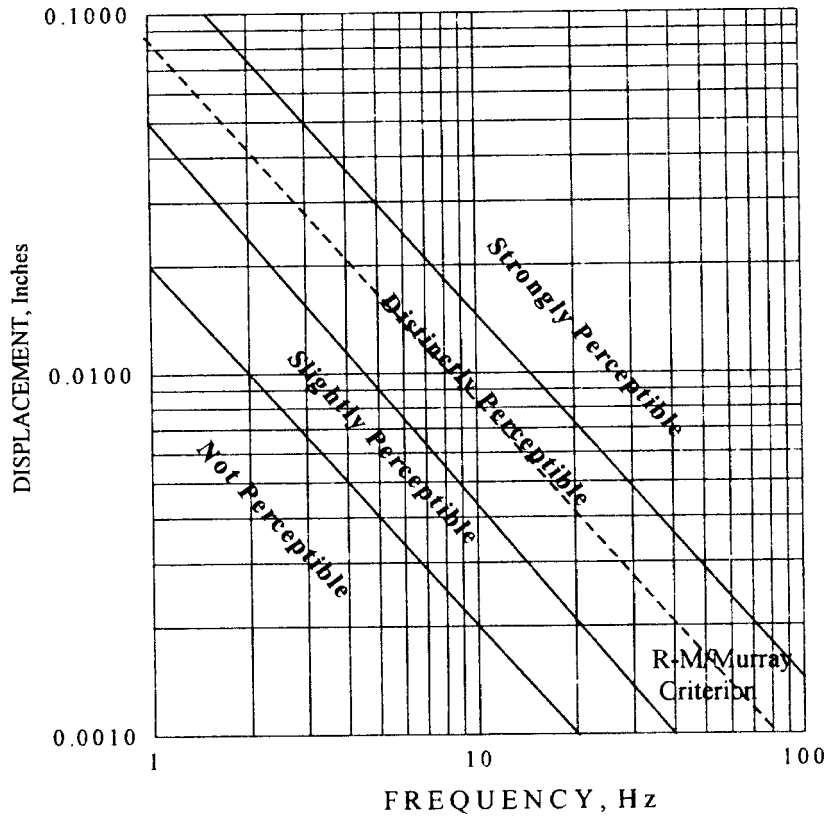


Figure 1. 4—Modified Reiher-Meister Scale

and the displacement used in the Modified Reiher-Meister scale was based on deflection from heel drop impact. The displacement, D , was estimated by

$$\Delta = \frac{300l^3}{48EI_t} \quad (1.2)$$

where I_t is as previous, g is acceleration of gravity (386.4 in./sec^2), E is the modulus of elasticity of steel, w_d is the dead load of the floor system, and l is the joist span. Here, Lenzen further accounted for the effects of damping by reducing the assumed impact from 600 lb (heel-drop) to 300 lb.

In 1981, Murray (1981) published the results of a study for which he compared the large number of criteria available at the time. He found significant disagreement; in several instances, the same floor was deemed acceptable by one criteria and unacceptable by another. However, upon examining results from 91 field tests on steel joist and steel beam-concrete slab floor systems, he found that levels of damping tend to separate the acceptable from unacceptable systems. Thus, the Murray Criterion established acceptability in terms of required

damping, stating that floors with fundamental frequencies less than 10 Hz and spans less than 40 ft will be acceptable if

$$D > 35A_0f + 2.5 \quad (1.3)$$

where D is percent critical damping, A_0 is the initial amplitude from a heel-drop impact, and f is the fundamental frequency.

Murray referred to his 1975 report (Murray 1975) as well as other available reports for estimation of the three parameters required for evaluation by the Murray Criterion. In that 1975 report, Murray suggested a method for estimating frequency that modified the previous Lenzen (1966) method. The supported weight, w , (w_d in Lenzen) was modified to include an estimate of actual live loading. The estimate should be at times when live loads are expected to be least. In addition, he included the Tee-beam model in Figure 1.5 for calculation of I_t , where the effective depth, d_e , can be used in lieu of calculations performed on the non-rectangular section. The heel drop method of estimating A_0 suggested by Murray (1975) will not be discussed here as the current method is based on walking rather than heel drop loading. The current method is discussed in Section 1.3.2. In subsequent research, Murray suggested that two-way behavior be account for in estimations of frequency. This concept is also illustrated in Section 1.3.2.

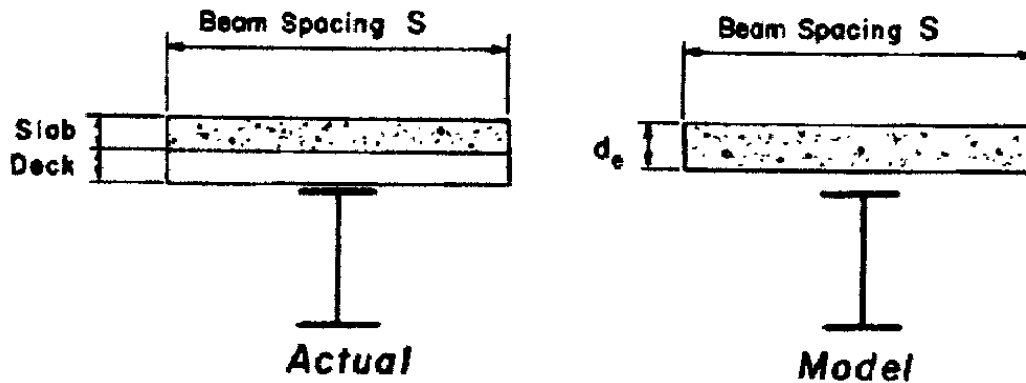


Figure 1. 5—Tee-beam Model (Murray 1975)

In 1993, Murray and Allen published *Design Criterion for Vibrations Due to Walking*. The criteria incorporate many of the previously discussed results. However, two major changes were made. First, walking impact replaces the heel drop as the basis for estimation of floor response. Second, the criteria account for both joist and girder modes of vibration; previously, only the dominant (lower frequency) mode was considered. The current walking excitation criteria of the *AISC Design Guide 11* are based on the 1993 Murray and Allen Cri-

teria.

1.3.2 Current Floor Vibrations Criteria: AISC DESIGN GUIDE 11

Murray, Allen, and Ungar (1997) authored the criteria that are referenced most often in this research. The criteria include guidelines for the design of office, residential, church, and footbridge structures as well as provisions for special considerations such as cantilevers and mezzanines. A general discussion of the criteria for walking excitation follows.

Evaluation. Acceptability of each floor bay is determined by comparing the peak acceleration to the acceptable acceleration limits shown in Figure 1.6 (0.5 % for office floors). The limits are as recommended by the International Standards Organization (ISO 2631-2: 1989).

The peak acceleration from walking excitation, a_p , expressed as a fraction of the acceleration of gravity, g , is

$$\frac{a_p}{g} = \frac{P_o e^{-0.35 f_n}}{\beta W} \quad (1.4)$$

where P_o is the constant force (65 lb for office floors), f_n is the system fundamental frequency, W is the system weight, and β is the modal damping ratio. Modal damping can be estimated as follows: 0.02 for floors with few non-structural components; 0.03 for floors with non-structural components, furnishings, and small demountable partitions; 0.05 for floors with full height partitions.

The weight supported, floor stiffness, and floor frequency must be estimated prior to determination of peak acceleration. Two dominant modes of vibration will typically be found in a floor bay. This means the joists will vibrate at their natural frequency, as will the girders. Thus, the weight, stiffness, and frequency calculations are performed for the joist (or beam) panel, the girder panel, then combined for the final evaluation.

Panel Weights. The weight of the joist panel, W_j , is estimated by

$$W_j = w_j B_j L_j \quad (1.5)$$

and the weight of the girder panel, W_g , is estimated by

$$W_g = w_g B_g L_g \quad (1.6)$$

where w is the supported weight per unit area, L is the length of the member, and B is the ef-

fective width of the floor. In addition to self-weight, the supported weight is to include 11 psf live load, which is typical of actual loading present on office floors. A panel weight may be increased by fifty percent if the panel members are continuously connected to a span at least 70% as long.

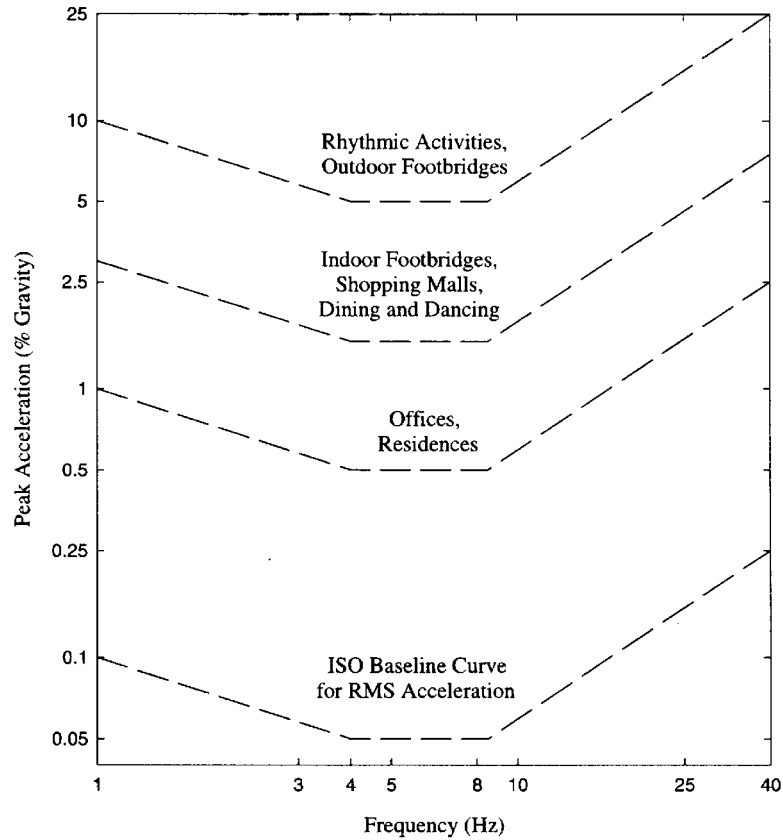


Figure 1. 6—Recommended Peak Acceleration

The effective widths, B_j and B_g , are determined as follows:

For the joist panel

$$B_j = 2(D_s / D_j)^{0.25} L_j \quad (1.7)$$

and for the girder panel

$$B_g = C_g(D_j / D_g)^{0.25} L_g \quad (1.8)$$

where C_g is 1.6 when joist seats are used or 1.8 when beams are used, D_j and D_g are the joist and girder transformed moments of inertia per unit width, and D_s is the slab moment of inertia per unit width.

Joist and Girder Stiffness. Estimated stiffness is used to determine the static deflection from the *panel weight*. The static deflection of the joist panel, D_j , is

$$\Delta_j = \frac{5w_j L_j^4}{384E_s I_j} \quad (1.9)$$

where w_j and L_j are as previous and E_s is the modulus of elasticity of steel. When hot-rolled sections or “beams” are used in the joist panel, I_j is taken as the fully composite moment of inertial. With joists, an effective moment of inertia must be calculated to account for shear deformations. Here, effective joist moment of inertia, I_{eff} , is

$$I_{eff} = 1 / [\gamma / I_{chords} + 1 / I_{comp}] \quad (1.10)$$

where I_{chords} is the moment of inertia of the joist chords only and I_{comp} is the composite moment of inertia of the slab and joist chords. The reduction factor, γ , is

$$\gamma = \frac{1}{C_r} - 1 \quad (1.11)$$

With joists having angle web members and span to depth satisfying $6 \leq L/D \leq 24$

$$C_r = 0.90(1 - e^{-0.28(L/D)})^{2.8} \quad (1.12)$$

and for joists having rod web members and span to depth satisfying $10 \leq L/D \leq 24$

$$C_r = 0.721 + 0.00725(L/D) \quad (1.13)$$

Similarly, girder panel static deflection, D_g , is taken as

$$\Delta_g = \frac{5w_g L_g^4}{384E_s I_g} \quad (1.14)$$

Here, girder full composite moment of inertia is used if the girders are in direct contact with the slab. This is acceptable for occupant induced vibration because deflections are small enough that friction compositely joins the components. However, when seated joists support the slab, the height of the joist seats separates the girder and slab, and the horizontal shear must be resisted by the joist seats rather than by friction. Here, the effective girder moment of inertia is taken as

$$I_{eff} = I_g + (I_{comp} - I_g) / 4 \quad (1.15)$$

where I_g is the bare girder moment of inertia, and I_{comp} is the girder full composite moment of inertia. If joist-girders are used, I_g of Equation 1.14 shall be taken as $I_g = C_r I_{chords}$ with C_r

calculated as previous.

Panel Frequency. The fundamental frequency for the joist panel is

$$f_j = 0.18 \sqrt{\frac{g}{\Delta_j}} \quad (1.16)$$

and for the girder panel

$$f_g = 0.18 \sqrt{\frac{g}{\Delta_g}} \quad (1.17)$$

where g is the acceleration of gravity (386 in/s²).

Combined Mode. After determining frequencies for the two panels, the system frequency, f_n , can be estimated with Dunkerley's relationship:

$$\frac{1}{f_n^2} = \frac{1}{f_j^2} + \frac{1}{f_g^2} \quad (1.18)$$

Expressed in terms of panel deflections, the relationship is

$$f_n = 0.18 \sqrt{\frac{g}{\Delta_j + \Delta_g}} \quad (1.19)$$

and the combined weight, W , can be estimated by

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g \quad (1.20)$$

1.3.3 Previous Research

As stated in the previous section, the current Design Guide provisions for effective moment of inertia of joist-girders and girders with joist seats as horizontal shear connections assume that the effective moment of inertia is equal to

$$I_{eff} = I_g + (I_{comp} - I_g) / 4 \quad (1.15)$$

This relationship was developed from single bay tests with joists with 2.5 in. seats, spaced 24-30 in. on center and supported by hot-rolled girders. However, research on the subject has continued.

Kitterman (1994) conducted several tests on a single 30 ft test floor with K-series

joists at 40 in. on center and supported by joist-girders. The composite action predicted by Equation 1.15 is $\frac{1}{4}$ or 25%; Kitterman measured no more than 20% composite action. However, the results were mixed as to the exact amount.

Kitterman and several others (Band 1996, Sladki 1999, and Alvis 2001) have had some success using finite element analysis (FEA) to predict floor stiffness and frequency. However, problems have arisen in predicting peak acceleration with the models. FE models will not be used for prediction of peak acceleration in this research.

Kitterman and Sladki considered the composite effect of joist seats in their FE models. Kitterman did so by reducing the stiffness of various members to attain measured or predicted stiffness. However, he did not attempt to model seat behavior. Sladki did model seat behavior in a limited manner. He compared FE model frequencies with field measurements of frequency from several buildings. He modeled joists seats using an arbitrarily chosen 0.001 in.^4 moment of inertia for seat elements. Table 1.1 shows results from the four Sladki tests that apply to the joist seats problem. Models of each of the four floors predicted very nearly the same frequency as the AISC design guide. The models of floors with K-joist, which have 2.5 in. tall seats, matched measured frequency or were slightly conservative. However, both models of floors with LH-joists, which have 5 in. tall seats, overestimated the measured frequency. The floors with LH-joists were likely more flexible than predicted by both the Design Guide and the FE modeling.

Table 1. 1—Results from Sladki (1999)

Framing	FREQUENCY, Hz		
	Measured	FEM	AISC
LH-joists at 36 in. on 31 ft Joist-Girders	6.75	7.66	7.11
LH-Joists at 72 in. on 24 ft HR-Girders	4.25	4.4	4.47
K-Joists at 29 in. on 32 ft Josit-Girders	3.75	3.71	3.70
KSP-Joists at 34 in. on 20 ft HR-Girders	5.75	5.73	5.58

1.4 Need For Research

Joist-girder/Girder Effective Moment of Inertia. Prediction of floor stiffness is essential in the evaluation of a floor for vibrations. It is essential because frequency is a function

of stiffness and acceptability is a function of frequency. Before the last few decades, the effective moment of inertia of joist-girders and girders with joist seats as horizontal shear connections was commonly assumed to be the full composite moment of inertia. This assumption was largely unconservative and was greatly improved in the AISC/CISC Design Guide 11, which predicts 25% composite action.

However, the design of floor systems with widely spaced LH-joists has become common. This is of concern because the Design Guide relationship does not account for spacing wider than 30 in. or joist seats taller than 2.5 in. As shown by Kitterman (1994), the Design Guide relationship, based on 24-30 in. spacing, is unconservative for 40 in. joist spacing. As indicated by Sladki (1999), 5 in. joist seats (LH-joists) may provide less shear connection than 2.5 in joist seats (K-joists). Thus, it is evident that research is needed to define the effects of wider spacing and taller seat heights, which may be leading to more flexible girders than predicted with the Design Guide relationship.

This research is the subject of Chapter 2. Eight 30 ft x 8 ft “footbridges” with girders or joist-girders spanning the 30 ft dimension and joists transverse to the girders, were tested. Joist spacing and joist type were varied between the floors. For each floor, three seat to girder connection types were tested: bolted, welded, and reinforced. Assessment of the effective moment of inertia was made using results from static, dynamic, and seat stiffness testing and FE modeling.

Cost of Composite and Non-Composite Floors. Recently, composite construction has been used to improve cost efficiency by reducing structural weight and in some cases by reducing story height. However, vibration problems are a design consideration in composite floors because lighter floors tend to be more lively. To determine which type construction is more economical when vibration criteria are met, four typical size bays were considered: 28 ft x 40 ft, 30 ft x 40 ft, 30 ft x 30 ft, and 40 ft x 40 ft. Using the design software “SDI FLOOR” (Elhouar and Murray 1993), all acceptable configurations of member sizes and spacing were evaluated for composite and non-composite cost. Three separate trials were conducted, with initial dead load deflection limits and unit stud cost as the variables. This study is the subject of Chapter 3.

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