

STUDY TO IMPROVE THE PREDICTED RESPONSE OF FLOOR SYSTEMS DUE TO WALKING

by

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(ABSTRACT)

The scope of this study is divided into three topics. To begin, more accurate methods for estimating the fundamental natural frequencies of floors were explored. Improvements for predicting the behavior of floor systems using several criteria were also investigated. The final topic compared the AISC and SCI methods for analyzing vibrations acceptability.

Natural frequency prediction was studied by examining 103 case studies involving floor systems of various framing occupied or being constructed in the United States and Europe. Based on the results from these comparisons, it was reasonably concluded that the predicted bay frequency using Dunkerly's estimate (f_n^2) is not the most accurate method for predicting the system frequency using the AISC Design Guide for all types of framing analyzed. The predicted beam frequency using AISC methods provided sound correlations with the measured bay frequencies. On the other hand, with the exception of floor systems with joist girders and joists, the results showed that the SCI methods provided more accurate predictions of bay frequency despite a fair amount of data scatter.

Evaluations based on the AISC Design Guide 11, the SCI criteria Murray Criterion, and Modified Reiher-Meister scale were compared with subjective field analyses for each case study in the second part of this study. The AISC Design Guide criterion is the most consistent method for predicting floor behavior. The SCI criterion is

the next most consistent method for floor acceptability, followed by the Murray Criterion then the Modified Reiher-Meister scale.

In the final part of this study, predicted accelerations and floor behavior tolerability for 78 case studies were evaluated using the AISC and the SCI criteria. The two prediction methods are in agreement for 82 % (64 of 78) of the case studies, and strongly disagree for only 12 % (9 of 78) of the case studies.

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TABLE OF CONTENTS

	PAGE
ACKNOWLEDGMENTS.....	iv
TABLE OF CONTENTS	v
LIST OF FIGURES.....	viii
LIST OF TABLES	x
CHAPTER I – FLOOR VIBRATIONS BACKGROUND AND LITERATURE REVIEW.....	1
1.1 INTRODUCTION	1
1.2 SCOPE OF RESEARCH.....	2
1.3 TERMINOLOGY	2
1.4 LITERATURE REVIEW	6
1.4.1 HISTORICAL FLOOR VIBRATIONS CRITERIA.....	6
1.4.2 AISC STEEL DESIGN GUIDE SERIES 11	10
1.4.3 SCI FLOOR VIBRATIONS CRITERIA.....	15
1.4.4 ADDITIONAL CURRENT RESEARCH	20
1.5 NEED FOR RESEARCH	21
CHAPTER II – NATURAL FREQUENCY STUDY	23
2.1 INTRODUCTION	23
2.2 AISC FREQUENCY COMPARISONS.....	24
2.2.1 FLOORS WITH HOT-ROLLED SHAPES.....	25
2.2.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS	28
2.2.3 FLOORS WITH JOIST GIRDERS AND JOISTS	30
2.2.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS	33
2.2.5 SUMMARY OF AISC FREQUENCY COMPARISONS	35
2.3 SCI FREQUENCY COMPARISONS.....	37
2.3.1 FLOORS WITH HOT-ROLLED SHAPES.....	37
2.3.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS	40
2.3.3 FLOORS WITH JOIST GIRDERS AND JOISTS.....	43
2.3.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS	45

2.3.5	SUMMARY OF SCI FREQUENCY COMPARISONS	47
2.4	AISC vs. SCI FREQUENCY COMPARISON	49
2.4.1	FLOORS WITH HOT-ROLLED SHAPES.....	49
2.4.2	FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS	49
2.4.3	FLOORS WITH JOIST GIRDERS AND JOISTS.....	49
2.4.4	HOT-ROLLED GIRDERS AND CASTELLATED BEAMS	50
2.5	FREQUENCY EVALUATION OF BRITISH FLOORS.....	53
2.5.1	AISC FREQUENCY EVALUATION OF BRITISH FLOORS	53
2.5.2	SCI FREQUENCY EVALUATION OF BRITISH FLOORS	56
2.5.3	EXAMPLE EVALUATION OF A SCI TESTED FLOOR	57
2.6	SUMMARY	59
CHAPTER III – FLOOR VIBRATIONS CRITERIA COMPARISONS ...		64
3.1	INTRODUCTION	64
3.2	CRITERIA COMPARISON RESULTS	65
3.2.1	FLOORS WITH HOT-ROLLED SHAPES.....	66
3.2.2	FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS	66
3.2.3	FLOORS WITH JOIST GIRDERS AND JOISTS.....	70
3.3	SUMMARY	70
CHAPTER IV – AISC AND SCI ACCELERATIONS COMPARISON ...		72
4.1	INTRODUCTION	72
4.2	AISC VS. SCI ACCEPTABILITY COMPARISON.....	73
4.2.1	FLOORS WITH HOT-ROLLED SHAPES.....	74
4.2.2	FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS	75
4.2.3	FLOORS WITH JOIST GIRDERS AND JOISTS.....	77
4.2.4	HOT-ROLLED GIRDERS AND CASTELLATED BEAMS	78
4.3	SUMMARY	78
CHAPTER V – CONCLUSIONS AND RECOMMENDATIONS		80
5.1	SUMMARY	80
5.2	CONCLUSIONS.....	80
5.2.1	NATURAL FREQUENCY STUDY	80
5.2.2	FLOOR VIBRATION CRITERIA COMPARISON.....	81

5.2.3	AISC AND SCI ACCELERATION COMPARISONS	81
5.3	COMMENTS	81
5.4	AREAS FOR FURTHER RESEARCH.....	82
	LIST OF REFERENCES	84
	APPENDIX A – EXAMPLE CALCULATIONS	86
	APPENDIX A.1 AISC DESIGN GUIDE 11 CALCULATIONS.....	87
A.1.1	HOT-ROLLED SHAPES	88
A.1.2	HOT-ROLLED GIRDERS WITH JOISTS	92
A.1.3	JOIST GIRDERS WITH JOISTS	97
A.1.4	HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS.....	101
A.1.5	BRITISH FLOOR SYSTEM	105
	APPENDIX A.2 SCI FREQUENCY CALCULATIONS.....	109
A.2.1	HOT-ROLLED SHAPES	110
A.2.2	HOT-ROLLED GIRDERS WITH JOISTS	115
A.2.3	JOIST GIRDERS WITH JOISTS.....	120
A.2.4	HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS.....	125
A.2.5	BRITISH FLOOR SYSTEM	130
	APPENDIX A.3 SCI ACCELERATION CALCULATIONS	137
A.3.1	HOT-ROLLED SHAPES	138
A.3.2	HOT-ROLLED GIRDERS WITH JOISTS	139
A.3.3	JOIST GIRDERS WITH JOISTS.....	140
A.3.4	HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS.....	141
	APPENDIX A.4 MURRAY CRITERION CALCULATIONS	142
A.4.1	HOT-ROLLED SHAPES	143
A.4.2	HOT-ROLLED GIRDERS WITH JOISTS	148
A.4.3	JOIST GIRDERS WITH JOISTS.....	153
	APPENDIX A.5 MODIFIED REIHER-MEISTER CALCULATIONS.....	158
A.5.1	HOT-ROLLED SHAPES	159
A.5.2	HOT-ROLLED GIRDERS WITH JOISTS	164
A.5.3	JOIST GIRDERS WITH JOISTS.....	168
	VITA	173

LIST OF FIGURES

Figure 1.1 – Amplitude and Period.....	3
Figure 1.2 – Dynamic Loads (AISC Steel Design Guide Series 11 1997)	4
Figure 1.3 – Typical Mode Shapes for a Beam and Floor System	5
Figure 1.4 – Frequency Spectrum	6
Figure 1.5 – Reiher-Meister Scale	7
Figure 1.6 – Modified Reiher-Meister Scale	7
Figure 1.7 – T-Beam Model.....	8
Figure 1.8 – Recommended Peak Accelerations (Allen and Murray 1993)	10
Figure 1.9 – Modal Deflection of Beams.....	17
Figure 1.10 – Mode Shape Governed by (a) Secondary Beam Flexibility (b) Primary Beam Flexibility (Hicks, et al. 2000).....	18
Figure 2.1 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Shapes	27
Figure 2.2 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Girders and Joists	29
Figure 2.3 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Joist Girders and Joists.....	32
Figure 2.4 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Girders with Castellated Beams.....	34
Figure 2.5 – Measured Bay Frequencies vs. AISC Predicted Frequencies for All Framing Types.....	36
Figure 2.6 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Hot-Rolled Shapes	39
Figure 2.7 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Hot-Rolled Girders and Joists.....	42
Figure 2.8 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Joist Girders and Joists.....	44

Figure 2.9 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Hot-Rolled Girders and Castellated Beams	46
Figure 2.10 – Measured Bay Frequencies vs. SCI Predicted Frequencies for All Framing Types.....	48
Figure 2.11 – Measured Bay Frequency vs. Predicted AISC and SCI Bay Frequency for British Tested Floors.....	55
Figure 2.12 – First Floor Framing Plan SCI Headquarters (Hicks, et al. 2000)	58

LIST OF TABLES

Table 1.1 – L_{eff} and S Dimensions (Hicks, et al. 2000).....	20
Table 2.1 – AISC Frequency Comparison for Floors with Hot-Rolled Shapes.....	26
Table 2.2 – AISC Frequency Comparison for Hot-Rolled Girders and Joists.....	28
Table 2.3 – AISC Frequency Comparison for Floors with Joist Girders and Joists.....	31
Table 2.4 – AISC Frequency Comparison for Floors with Hot-Rolled Girders and Castellated Beams	33
Table 2.5 – SCI Frequency Comparison for Floors with Hot-Rolled Shapes	38
Table 2.6 – SCI Frequency Comparison for Floors with Hot-Rolled Girders and Joists .	41
Table 2.7 – SCI Frequency Comparison for Floors with Joist Girders and Joists.....	43
Table 2.8 – SCI Frequency Comparison for Floors with Hot-Rolled Girders and Castellated Beams	45
Table 2.9 – AISC vs. SCI Frequency Comparison for Floors with Hot-Rolled Shapes...	50
Table 2.10 – AISC vs. SCI Frequencies for Hot-Rolled Girders and Joists.....	51
Table 2.11 – AISC vs. SCI Frequencies for Floors with Joist Girders and Joists	52
Table 2.12 – AISC vs. SCI Frequencies for Hot-Rolled Girders with Castellated Beams	52
Table 2.13 – System Frequency Comparison for British Floors Using AISC Method	54
Table 2.14 – System Frequency Comparison for British Floors Using SCI Method	54
Table 2.15 – AISC vs. SCI Comparison of Parameters for the SCI Headquarters Framing	58
Table 2.16 – Summary of Results of Frequency Study	60
Table 3.1 – Acceptability Criteria Comparison for Floors with Hot-Rolled Shapes.....	67
Table 3.2 – Acceptability Criteria Comparison for Floors with Hot-Rolled Girders and Joists.....	68
Table 3.3 – Acceptability Criteria Comparison for Floors with Joist Girders and Joist Framing.....	69
Table 3.4 – Criteria Comparison Summary	71
Table 4.1 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Shapes.....	74
Table 4.2 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Girders and Joists.....	76

Table 4.3 – AISC vs. SCI Acceptability for Framing Using Joist Girders and Joists	77
Table 4.4 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Girders and Castellated Beams	78
Table 4.5 – AISC vs. SCI Acceptability Comparison Summary	79

CHAPTER I – FLOOR VIBRATIONS BACKGROUND AND LITERATURE REVIEW

1.1 INTRODUCTION

Current trends in technology and construction practices have had an adverse effect on the structural dynamics of floor systems in buildings. Twenty-first century building designs are lightweight and high-strength and have longer spanning floor systems. As a result, the serviceability due to vibrations becomes problematic for floors with less live loading, damping, stiffness, and mass. Although floor vibrations do not affect the structural integrity of a building, occupants may feel anxiety and uneasiness due to the motion. Research into floor vibrations must continue to evolve to keep up with changing developments in design and materials.

Floors that are susceptible to occupant-induced vibrations are costly and tedious to retrofit. A number of criteria and design standards have been developed to predict the response of floor systems due to walking excitations. Each criterion has assumptions on the nature and relevant properties of floor systems. Due to the assorted techniques, predictions of floor behavior can be significantly different between the criteria. Examining case studies of floor systems currently being constructed and occupied in the United States and the United Kingdom may provide insight on the accuracy of the various prediction criteria.

The focus of this research is divided into three main topics in which case studies were evaluated. The first focuses on comparing predicted natural frequencies with field-measured frequencies. The second is a comparison between current acceptability criteria and subjective field analyses. A comparison between the acceptability criteria used in the United States and the United Kingdom is assessed in the final topic. The results of each topic are used to suggest methods to improve correlation between predicted and field behavior.

1.2 SCOPE OF RESEARCH

The objective of this research is to examine case studies of floor systems of various framing being constructed and occupied in the United States and the United Kingdom in an attempt to improve correlation between predicted and measured floor response. Analyses of existing floors are important for improving the accuracy of techniques for predicting the fundamental natural frequency of a floor. The first topic provides a comparison between predicted fundamental frequencies and measured field frequencies. The second topic involves evaluating currently occupied floor systems using the American Institute of Steel Construction Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997), the Modified Reiher-Meister scale, the Murray Criterion, and the Steel Construction Institute criteria. The results of the evaluations are then compared to subjective field analyses of the floor systems. The third focus of this study compares the predicted accelerations and acceptability criteria between the American Institute of Steel Construction (AISC) and Steel Construction Institute (SCI) methods for assessing floor behavior. Proposals for continued research and conclusions on each specific study are then given.

1.3 TERMINOLOGY

The terms listed below are used frequently within the text. The terms are defined with respect to the specific field of floor vibrations.

Vibration. *Vibrations* are periodic motions of an elastic body such as a floor system in alternately opposite directions from a position of equilibrium. Floor vibrations can be classified as free or forced. *Free vibration* occurs when the floor system is displaced and released, causing the structure to vibrate at a natural frequency. *Forced vibration* results when a floor system vibrates at a frequency caused by a continuous and dynamic excitation.

Cycle. A *cycle* is the motion of a floor system that recurs regularly and usually leads back to a starting position.

Amplitude. *Amplitude* is the extent of a vibratory motion measured from a mean position to an extreme. The amplitude throughout the text will often be expressed as a percentage of gravity for floor accelerations. See Figure 1.1.

Period. *Period, T* , is an interval of time required for an oscillating motion to complete a cycle, typically measured in seconds. See Figure 1.1.

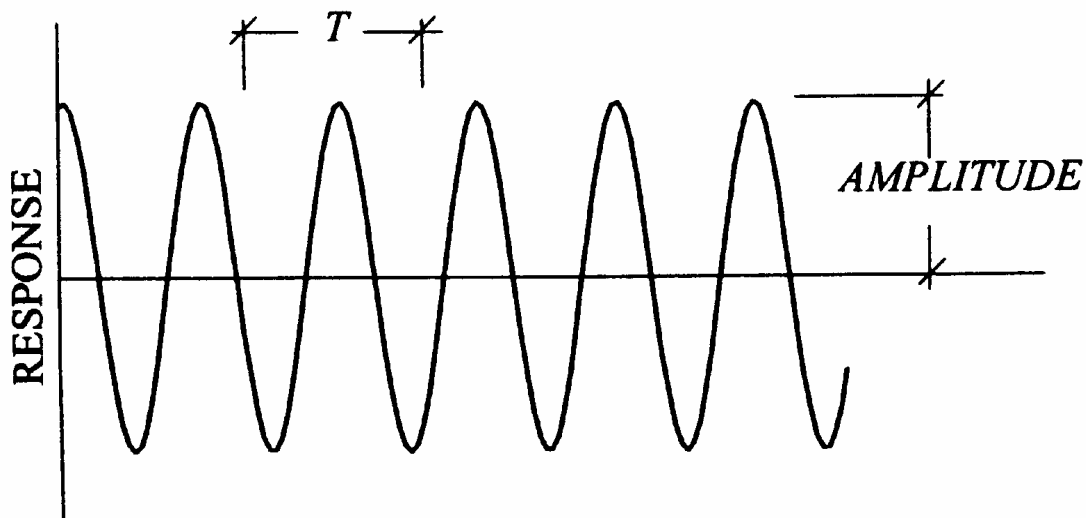


Figure 1.1 – Amplitude and Period

Frequency. *Frequency* is the inverse of the period or the quantity of cycles that occur during a certain unit of time. Frequency is expressed in *Hertz* (Hz) or the number of cycles per second. *Natural frequencies* occur during free vibration of a system and exhibit considerable amplitudes. The *fundamental natural frequency* is the lowest natural frequency of a floor system.

Damping. *Damping* is the process through which the vibration of a floor system diminishes in amplitude due to loss in mechanical energy. Damping that is proportional to velocity, *viscous damping*, is assumed. *Critical damping* is the minimum viscous damping required to prevent oscillation of a system. Damping throughout the text will be expressed as a percent of the critical damping.

Resonance. *Resonance* is a state that results in vibrations with large amplitudes as a result of an excitation applied to a system at any natural frequency or multiple of any natural frequency to the system.

Loading. *Dynamic loads* are excitations that change with respect to time. Loads induced by walking excitations are *transient loads*. Loads caused by rhythmic human activities are *periodic loads*. Loads caused by sudden impacts such as single jumps are referred to as *impulsive loads*. See Figure 1.2.

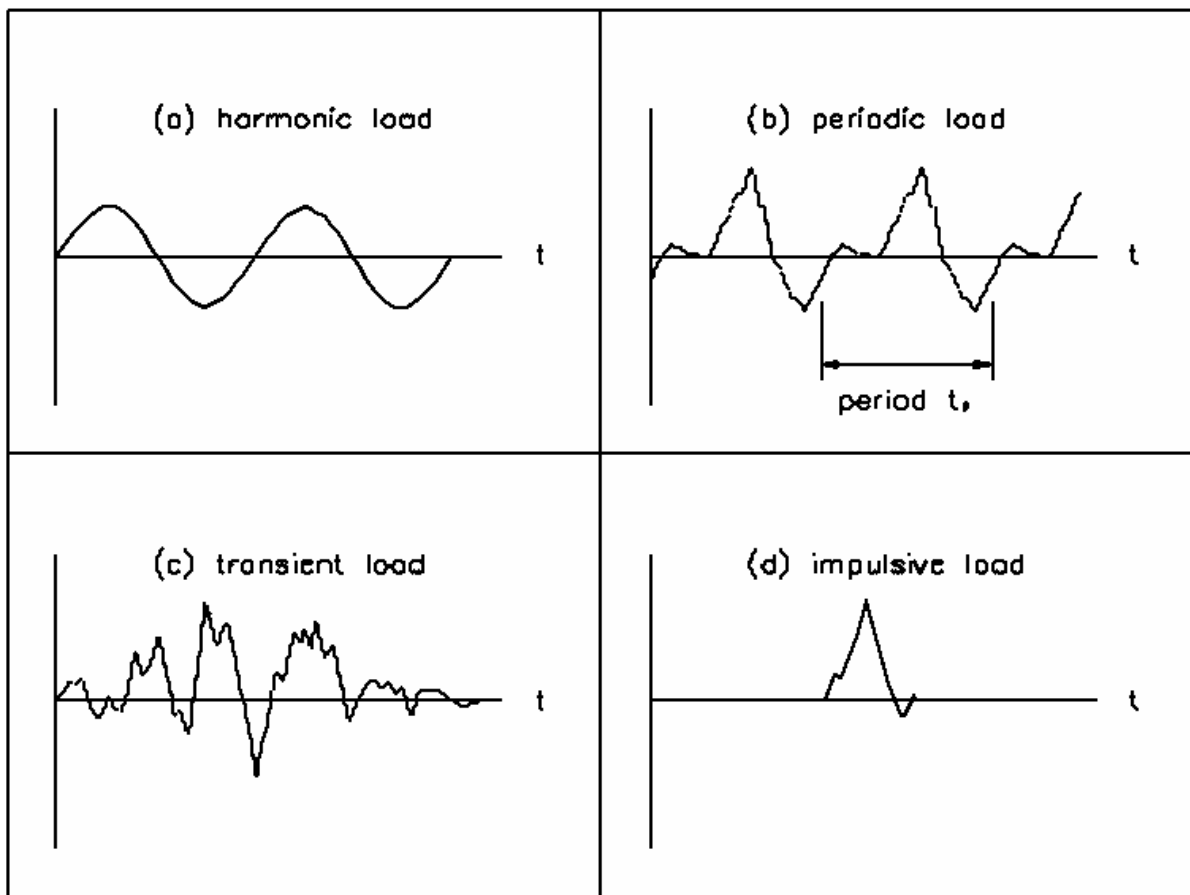


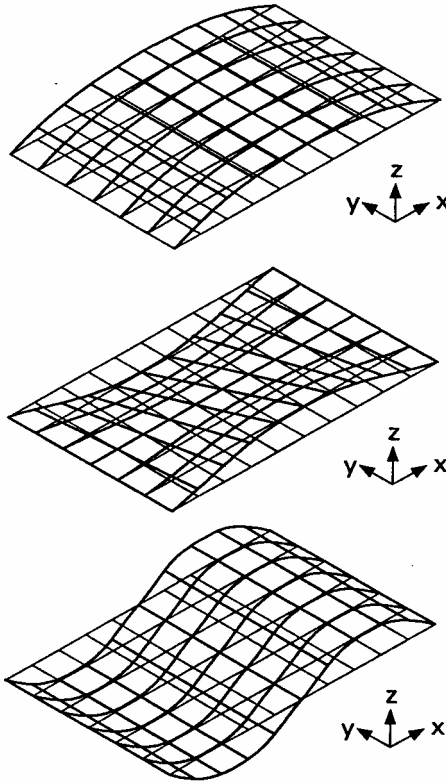
Figure 1.2 – Dynamic Loads (AISC Steel Design Guide Series 11 1997)

Heel Drop. A *heel drop* is an example of an impulsive load. The “standard heel drop” is defined as the force caused by the heels of a 170 lb person impacting the floor after a fall of approximately two and a half inches from a raised position.

Mode Shape. The *mode shape* represents the motion of a structure when vibrating at a natural frequency. See Figure 1.4 for mode shapes for beams and floor systems.



a) Beam



b) Floor System

Figure 1.3 – Typical Mode Shapes for a Beam and Floor System

Fast Fourier Transformation (FFT). The *FFT* is a mathematical method to transform a set of time dependent variables into a frequency spectrum. Spikes in the frequency spectrum represent the various modes of vibration for a floor system. See Figure 1.4.

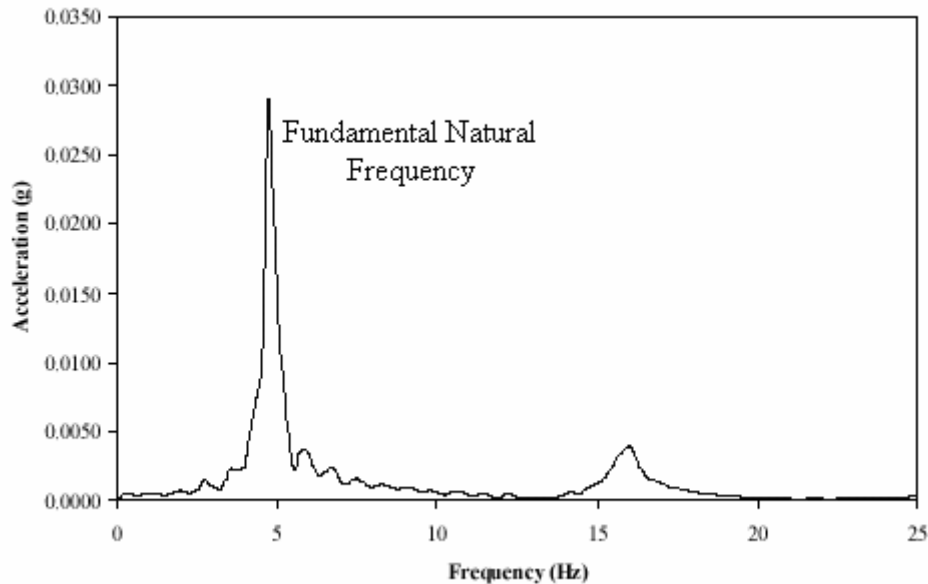


Figure 1.4 – Frequency Spectrum

1.4 LITERATURE REVIEW

The following section offers a historical perspective on the research involving floor vibrations. The basis for much of the current research, AISC Steel Design Guide Series 11, is then discussed in Section 1.4.2. The design criteria currently used in the United Kingdom is presented in Section 1.4.3. In Section 1.4.4, recent research relating to floor vibrations not involved directly with this study are discussed to emphasize the direction and growth of this particular science.

1.4.1 HISTORICAL FLOOR VIBRATIONS CRITERIA

Through the pioneering work by Reiher and Meister (1931), floor vibrations design criteria began to evolve. The research by Reiher and Meister was based on the human ability to perceive motion. A group of people were subjected to steady state vibration and were asked to rate their observations. The people experienced frequencies between 5 Hz to 100 Hz and amplitudes between 0.0004 in. and 0.4 in. A scale, shown in Figure 1.5, was produced base on the perceptions felt by this subject group. The scale begins with “not perceptible”, followed by “slightly perceptible”, “distinctly perceptible”, and “strongly perceptible”, then “disturbing” and “very disturbing”.

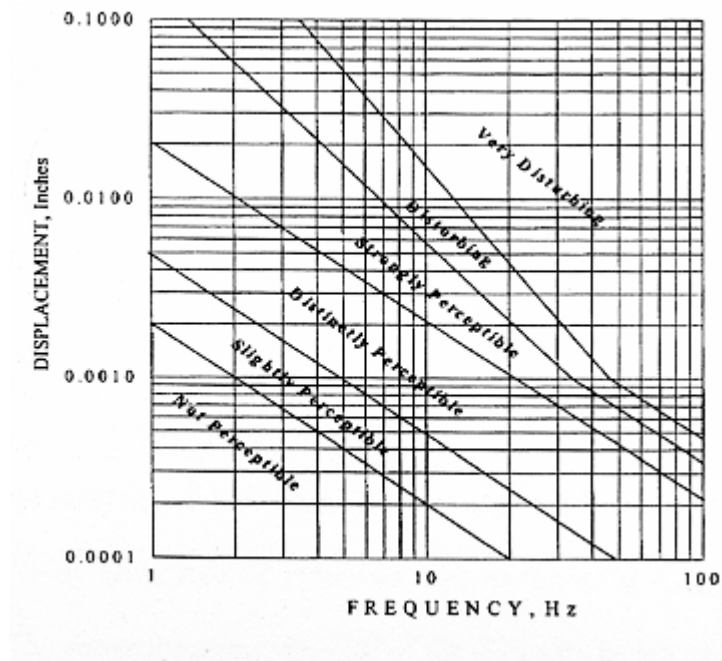


Figure 1.5 – Reiher-Meister Scale

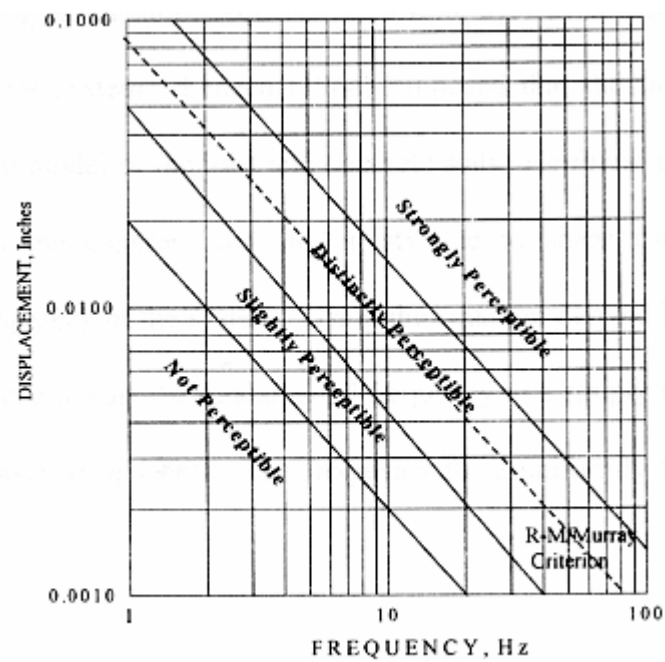


Figure 1.6 – Modified Reiher-Meister Scale

Floor vibrations were further investigated when Lenzen (1966) realized that humans are less perceptive to transient vibrations than steady state vibrations. Lenzen discovered that when damped transient vibrations reduce to negligible amplitudes compared to the initial amplitude within five cycles of motion, only the original impact would be sensed. Conversely, steady state vibration could be perceived if the motion continued for twelve or more cycles. Lenzen created the Modified Reiher-Meister scale; shown in Figure 1.6, by multiplying the displacement axis of the original Reiher-Meister scale by a factor of ten. Lenzen based this revision on conclusions that adequate damping found in typical floors will limit the magnitude of vibrations caused by transient loads.

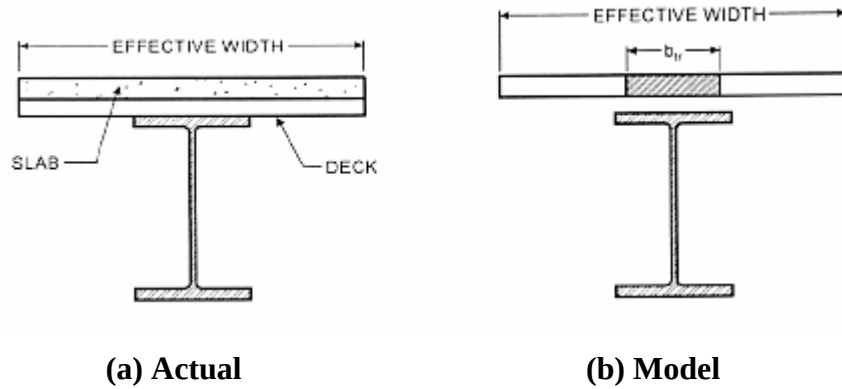


Figure 1.7 – T-Beam Model

In addition, Lenzen introduced methods for estimating floor system properties such as stiffness, frequency, and displacement. Lenzen used a T-beam model as shown in Figure 1.7 for the joists, assuming full composite action. The effective width for the model was the sum of half of the distances to adjacent joists. The system natural frequency, f , was found by

$$f = 1.57 \sqrt{\frac{gEI_t}{WL^3}} \quad (1.1)$$

where $g = 386 \text{ in/sec}^2$; E = modulus of elasticity for steel, ksi; I_t = sum of transformed moment of inertias for all joists, in^4 ; W = total weight supported by the T-beam, kips; and L = span of beam, in. The displacement, Δ , for use in Figure 1.6, is valid for spans up to 24 ft and is approximated by

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

where $P = 300$ lb for floors that have partitions; I_t , E , and L are as previously described. This displacement equation was based on impulsive loads such as heel drops and developed for use with the Modified Reiher-Meister scale.

The work of Murray (1975) contributed to further development of the Modified Reiher-Meister criteria. Murray concluded “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 dashed line on Figure 1.6 represents this amendment to the Modified Reiher-Meister scale. Furthermore, Murray recommended using an effective depth, d_e , for I_t calculations. The effective depth for a slab in the direction parallel to the beam is taken as the depth of the concrete above the deck. The effective depth for a slab in the direction perpendicular to the beam is taken as the sum of concrete depth and one-half the deck height.

Murray (1981) proposed the Murray Criterion by setting a minimum damping as a requirement for acceptability. The required percent critical damping, D , is given by

$$D > 35A_o f + 2.5 \quad (1.3)$$

where f = first natural frequency, Hz, and A_o = initial amplitude based on heel drop approximations suggested by Murray (1975). Only floor systems that experienced frequencies less than 10 Hz and with spans less than 40 ft were applicable to the criteria. Murray based his recommendation on a statistical analysis of 91 field tests and concluded that considerable disagreement between various floor vibrations criteria was a result of discrepancies in damping.

Design criteria for floor vibrations based on walking excitations rather than heel drop excitations were first initiated through the work of Allen and Rainer (1976). The criterion developed was integrated into the Canadian Standards Association Standard, CSA S16.1 (CSA, 1989).

In 1989, The International Standards Organization (ISO) suggested limits in terms of root-mean-square (rms) acceleration as a multiple of the baseline curve, shown in Figure 1.8. Allen and Murray (1993) recommended peak accelerations, as shown in Figure 1.8, for offices, residences, malls, and footbridges in the publication *Design Criterion for Vibrations Due to Walking*. In addition, Allen and Murray developed

design techniques and approaches for estimating required floor properties based on walking excitations. Many of the same procedures used in the AISC Steel Design Guide Series 11 *Floor Vibrations Due to Human Activity*, referred henceforth as the Design Guide, is based on the work of Allen and Murray. The Design Guide is the most current criteria used by engineers in North America and was written by Murray, Allen, and Ungar (1997). Since most of this study is primarily based on the methods introduced in the Design Guide, the following section will provide a more detailed discussion of the walking criterion in this publication.

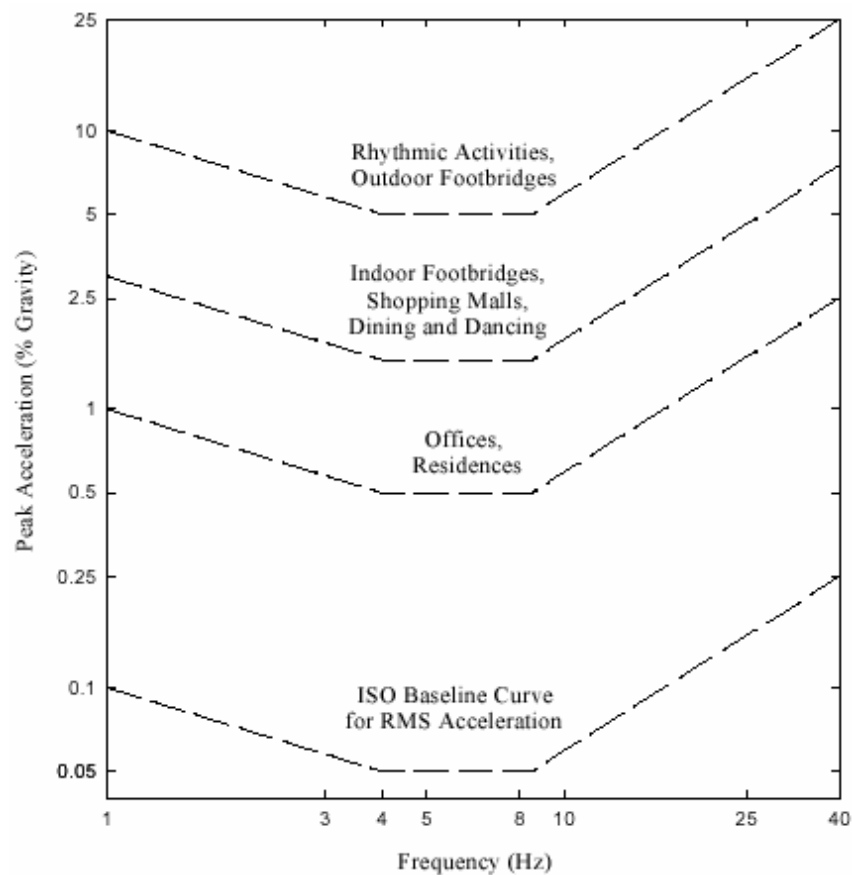


Figure 1.8 – Recommended Peak Accelerations (Allen and Murray 1993)

1.4.2 AISC STEEL DESIGN GUIDE SERIES 11

The dynamic response of steel beam and joist supported floor systems to walking excitations provides the foundation for the criterion in Chapter 4 of the Design Guide. The Design Guide criterion can be used to investigate structural systems of footbridges,

offices, residences, and malls made of steel and concrete. This section will explain the walking criterion and summarize the important procedures for calculating floor system properties.

Acceptability. The criterion states that a floor system is satisfactory “if the peak acceleration, a_p , due to human walking excitation as a fraction of the acceleration of gravity, g , determined from

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \leq \frac{a_o}{g} \quad (1.4)$$

where P_o = a constant force representing the excitation, f_n = fundamental natural frequency of a beam or joist panel, a girder panel, or combined panel, as applicable, β = modal damping ratio, and W = effective weight supported by the beam or joist panel, girder panel, or combined panel, as applicable, does not exceed the acceleration limit a_o/g for the appropriate occupancy”. The scope of this research covers the behavior of floors in office environments. Therefore, the acceleration limit, as shown in Figure 1.8, for office occupancies is taken as 0.5 % of gravity. The recommended value of P_o is 65 lbs for the same occupancy. Actual dead and live loads rather than design dead and live loads are assumed for weight calculations. A typical office super-imposed dead load of 4 psf, which accounts for ceiling and mechanical loads, is generally assumed in floor vibrations calculations. Damping and live loading values vary for different types of offices. This research investigates “paper” offices and “electronic” offices. The typical fit-out in a “paper” office includes heavy filing cabinets, desks, furniture, computers, and technical equipment. Consequently, typical assumed values for damping and live loading are 3 % and 11 psf, respectively. Lighter floor systems in “electronic” offices have typical assumed values for damping between 2 % to 2.5 % and live loads of 8 psf. The typical fit-out in an “electronic” office includes only desks and computers.

Fundamental Frequency. To begin evaluating a floor, the fundamental frequency of the joists and girders need to be ascertained. The fundamental frequency, f , for each framing member is estimated from

$$f_j = f_g = 0.18 \sqrt{\frac{g}{\Delta_{ss}}} \quad (1.5)$$

where f_j = frequency of beam or joist, Hz; f_g = frequency of girder, Hz; g = acceleration due gravity (386 in./s²) and Δ_{ss} = deflection of the joist, beam, or girder, in., due to the weight supported, w . Framing members in floor systems are assumed to be simply-supported. Therefore, the maximum deflection, Δ_{ss} , at midspan of a simply supported member is estimated by

$$\Delta_{ss} = \Delta_b = \Delta_g = \frac{5wL^4}{384E_s I} \quad (1.6)$$

where w = total supported weight including an estimation of live loading, kips/ft; E_s = modulus of elasticity for steel, ksi; L = member span, in.; and I = composite moment of inertia, in.⁴. When calculating I , the Design Guide advises that the effective width of the concrete slab be taken as the member spacing, but not more than 40 % of the member span. Also, when determining I , the concrete modulus of elasticity may be increased by 35 % to account for greater stiffness of the slab during dynamic loading. In addition, the type of beam is important when calculating I . The full composite moment of inertia is used if the beam is a hot-rolled section. However, joists experience considerable shear deformations unlike hot-rolled sections during loading, thus reducing the full composite moment of inertia. The smaller effective composite moment of inertia, I_{eff} , is estimated by

$$I_{eff} = \frac{1}{\frac{\gamma}{I_{chords}} + \frac{1}{I_{comp}}} \quad (1.7)$$

where I_{chords} = moment of inertia of the chords of the joist, in.⁴; I_{comp} = transformed moment of inertia of the chords and slab, in.⁴. The reduction factor, γ , was proposed by Band and Murray (1996) and is calculated by

$$\gamma = \frac{1}{C_r} - 1 \quad (1.8)$$

where, for joists or joist girders with angle web members and a span-to-depth ratio satisfying $6 \leq L/D \leq 24$, C_r is estimated by

$$C_r = 0.90(1 - e^{-0.28(L/D)})^{2.8} \quad (1.9)$$

and for joists with rod web members a span-to-depth ratio satisfying $10 \leq L/D \leq 24$, C_r is estimated by

$$C_r = 0.721 + 0.00725(L/D) \quad (1.10)$$

where L = span length and D = nominal depth of joist.

When calculating girder properties, the full transformed moment of inertia, I_{comp} , is used in the deflection equation (1.6) when girders are in direct contact with the slab and hot-rolled shapes are used as the secondary framing. However, when girders or joist-girders are supporting open web joists, an effective moment of inertia, I_{eff} , is used to account for the lack of stiffness provided by joist seat connections. The effective moment of inertia is calculated by

$$I_{eff} = I_g + (I_{comp} - I_g)/4 \quad (1.11)$$

where I_g = the moment of inertia of the bare girder and I_{comp} = full composite moment of inertia, in.⁴.

Once the beam and girder frequencies are determined, the combined frequency of the floor system can be calculated using Dunkerly's relationship given by

$$\frac{1}{f_n^2} = \frac{1}{f_j^2} + \frac{1}{f_g^2} \quad (1.12)$$

where f_n , f_j , and f_g are the bay system, beam, and girder frequencies, Hz, respectively. The system frequency can also be calculated by

$$f_n = 0.18 \sqrt{\frac{g}{\Delta_j + \Delta_g}} \quad (1.13)$$

which is another form of Dunkerly's equation.

Panel Weights. The effective panel weight for a beam panel mode, W_j , is estimated from

$$W_j = w_j B_j L_j \quad (1.14)$$

and similarly the effective panel weight for a girder panel mode, W_g , is estimated from

$$W_g = w_g B_g L_g \quad (1.15)$$

where w = supported weight, kips/ft²; B = effective width of panel, ft; L = span length of the member, ft. For the beam or joist panel mode, the effective width is calculated from

$$B_j = C_j (D_s/D_j)^{1/4} L_j \quad (1.16)$$

where $C_j = 2.0$ for joists and beams in most areas or $C_j = 1.0$ when joists or beams are parallel to an interior edge; D_j = joist or beam transformed moment of inertia per unit width, in.⁴/ft; D_s = transformed slab moment of inertia per unit width, in.⁴/ft; L_j = span length of the member, ft. The effective width of the beam or joist panel mode cannot be greater than two-thirds of the floor width in the direction perpendicular to the beam or joist span. For the girder mode, the effective width is calculated by

$$B_g = C_g (D_j / D_g)^{1/4} L_g \quad (1.17)$$

where $C_g = 1.6$ for girders supporting joists connected to the girder flange through joist seats or $C_g = 1.8$ for girders supporting beams connected to the girder web; D_j = joist or beam transformed moment of inertia per unit width, in.⁴/ft; D_g = girder transformed moment of inertia per unit width, in.⁴/ft; L_g = span length of the girder, ft. The effective width of the girder panel mode cannot be greater than two-thirds of the floor length in the direction perpendicular to the girder span. The effective panel weight, W_j or W_g , may be increased by 50 % when beams, joists, or girders are continuous over their supports and a neighboring span is greater than 0.7 times the span under consideration. This increase in weight applies to hot-rolled sections shear-connected to girder webs and joists that are connected by both top and bottom chords.

The equivalent panel weight, W , when considering the combined mode is estimated by

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g \quad (1.18)$$

where Δ_j , Δ_g , W_j , and W_g are as described previously. The deflection of the girder, Δ_g , can be reduced if the girder span, L_g , is less than the joist panel width, B_j . The floor system is stiffened and the combined mode is constrained, thus justifying the reduction. The reduced girder deflection, Δ_g' , is approximated by

$$\Delta_g' = \frac{L_g}{B_j} (\Delta_g) \quad (1.19)$$

where L_g/B_j is taken as not less than 0.5 nor greater than 1.0. The predicted natural frequency is increased with a reduction in the deflection of the girder. The deflection Δ_g

in Equations 1.5 or 1.13 is replaced by Δ_g' in order to calculate the fundamental natural frequencies of girders or system frequencies.

1.4.3 SCI FLOOR VIBRATIONS CRITERIA

The SCI Publication *Design Guide on the Vibration of Floors*, authored by Wyatt (1989), provides the techniques and criterion used in the United Kingdom for analyzing floor response. In this section, the acceptability criteria used to rate a floor system, the methods used to calculate the fundamental natural frequencies of floor systems, important procedures for calculating floor system properties, and a summary of the different floor system assumptions currently used by AISC and SCI techniques, are presented.

Acceptability Criteria. Using estimated peak acceleration, the root-mean-square acceleration or a_{rms} is then

$$a_{rms} = a_{peak} / \sqrt{2} \quad (1.20)$$

The limiting acceleration is given by the baseline acceleration, a_{base} , shown as the ISO baseline curve in Figure 1.8 multiplied by a response factor, R , for the specific office environment. The limiting acceleration is also a function of the fundamental frequency, f_o , of the floor system. The limiting accelerations of floors are dictated by

$$a_{rms} \leq 0.005R \quad \text{for } 3.0 \text{ Hz} \leq f_o \leq 8.0 \text{ Hz} \quad (1.21)$$

and

$$a_{rms} \leq 0.005R \left(\frac{f_o}{8} \right) \quad \text{for } f_o > 8.0 \text{ Hz} \quad (1.22)$$

The SCI guide recognizes three types of office environments. The “special office” is “appropriate for floors where technical tasks that require concentration and precision operations on computers are performed”. The “general office” is “appropriate for floors where normal activities and text operations on computers are performed”. The “busy office” is “appropriate for floors that are available to many people, with both visual and audible distractions”. The response factors for “special office”, “general office”, and “busy office” are 4, 8, and 12, respectively.

Fundamental Frequency. Many of the same procedures and calculations are used by both SCI and AISC. The Steel Construction Institute also uses Equations 1.5 and

1.6 to calculate the fundamental natural frequency of floors. However, SCI has subtle differences in the assumptions of floor system properties. For example, when calculating the weight carried by a beam, SCI assumes a load corresponding to the self-weight, services, ceiling, and 10 % of the imposed load. The imposed value of 10 % represents “a sensible permanent load for furnished floors designed in office environments”. The AISC and the SCI also differ in the definition of effective width for secondary and primary framing members. The SCI takes the effective breadth of slab to be the summation of one-eighth of the member spans either side of the beam providing that this value does not exceed b or the summation of $0.8b/2$, where b is the spacing of the secondary and primary framing, respectively.

The AISC and SCI guide allows for a reduction in the deflection of composite beams that have dissimilar spans and are continuous over supports. According to the AISC publication *Floor Vibrations Due to Human Activity* (1997), the reduced deflection, $\bar{\delta}$, of the main span for a floor with two continuous spans being considered is estimated by

$$\bar{\delta} = \left[\frac{0.4 + \frac{k_m}{k_s} \left(1 + 0.6 \frac{L_S^2}{L_M^2} \right)}{1 + \frac{k_m}{k_s}} \right] \Delta_{ss} \quad (1.23)$$

The reduced deflection, $\bar{\delta}$, of the main span for a floor with three continuous spans being considered is estimated by

$$\bar{\delta} = \left[\frac{0.6 + 2 \frac{k_m}{k_s} \left(1 + 1.2 \frac{L_S^2}{L_M^2} \right)}{3 + 2 \frac{k_m}{k_s}} \right] \Delta_{ss} \quad (1.24)$$

where $k_m = I_M/L_M$, $k_s = I_S/L_S$, I = moment of inertia, in.⁴; L_M and L_S are defined in Figure 1.9. The deflection Δ_{ss} in Equation 1.5 can be replaced by $\bar{\delta}$ to calculate the fundamental natural frequencies of beams or girders.

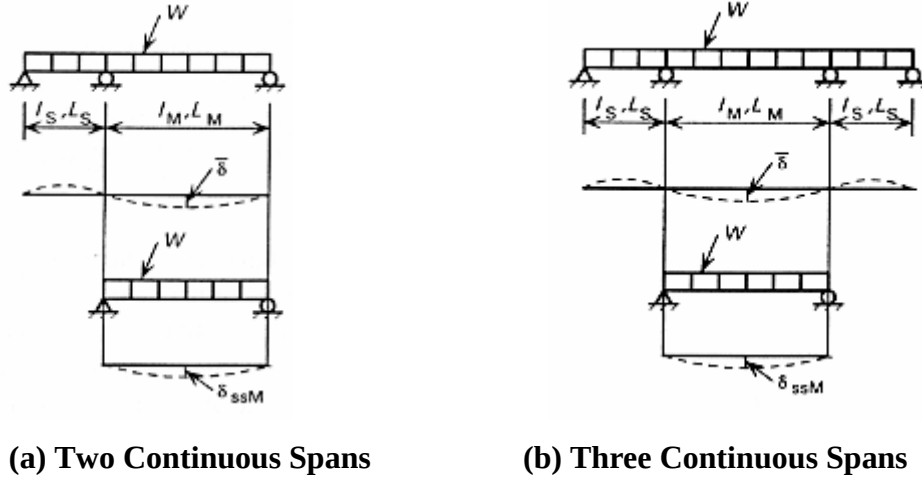


**Figure 1.9 – Modal Deflection of Beams
(AISC Design Guide 11 1997)**

Another difference between the AISC and SCI techniques is that SCI considers the deflection of the composite slab when calculating the fundamental frequency of the entire floor system. Dunkerly's approximation becomes

$$\frac{1}{f_n^2} = \frac{1}{f_j^2} + \frac{1}{f_g^2} + \frac{1}{f_{slab}^2} \quad (1.25)$$

where f_{slab} = frequency of composite slab, Hz; and f_n , f_j , and f_g are as described previously. In composite construction, the slab is much stiffer than the beam or girder. Therefore, in many analyses of floor systems, the frequency of the slab may be neglected.

The Steel Construction Institute considers two modes of vibration when investigating a structural floor system. As shown in Figure 1.10(a), when a floor system is controlled by the secondary beam mode, primary beams behave as nodal lines allowing no deflection. The secondary framing vibrates as simply-supported beams. Fixed-ended boundary conditions are assumed for slab frequency estimations. When floor systems are controlled by the primary beam mode, as shown in Figure 1.10(b), primary framing members vibrate about columns as simply-supported beams. Fixed-ended boundary conditions are assumed for secondary framing and slab frequency estimations. When the primary beam mode is considered, the deflection of the secondary beam, Δ_b , is calculated by

$$\Delta_b = \frac{WL^3}{384EI} \quad (1.26)$$

where W = total weight supported by the framing member, kips; E = modulus of elasticity for steel, ksi; L = member span, in.; and I = composite moment of inertia, in.⁴. Deflection of the primary beam, Δ_g , is calculated by Equation 1.16.

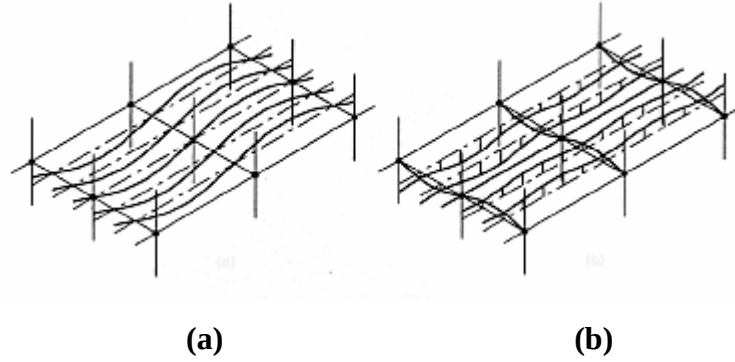


Figure 1.10 – Mode Shape Governed by (a) Secondary Beam Flexibility (b) Primary Beam Flexibility (Hicks, et al. 2000)

Peak Acceleration. The acceptability of vibrations induced by walking excitations is based on predicting the peak acceleration. Methods for calculating the peak acceleration are outlined in the *Design Guide for Vibrations of Long Span Composite Floors* (Hicks, et al. 2000). The peak acceleration, a_{peak} , measured in meters per second (m/s) assumes resonant response and is calculated by

$$a_{peak} = \frac{\alpha_n P}{2M\zeta} R_1 \quad (1.27)$$

where α_n = Fourier coefficient of the n^{th} harmonic component of the walking activity and P = person's weight take as 76 kg. The only Fourier coefficient that needs to be considered when the fundamental frequency of a floor system is greater than or equal to 3.55 Hz is $\alpha_n = 0.10$. This Fourier coefficient is “appropriate for walking frequencies greater than the first harmonic”. When evaluating floors with predicted frequencies of less than 3.55 Hz, the first harmonic is considered and $\alpha_n = 0.40$. The resonance build-up factor, R_1 , is given by

$$R_1 = 1 - e^{-2\pi f_o T_w} \leq 1.0 \quad (1.28)$$

where f_o = fundamental frequency of floor, Hz, and T_w = the walking time in seconds (s) calculated by

$$T_w = D/V \quad (1.29)$$

where D is taken in meters (m) as the longer of the floor's plan dimensions or, when known, the longest corridor length. The walking velocity, V , in m/s is calculated by

$$V = 1.67 f_p^2 - 4.83 f_p + 4.50 \quad \text{for } 1.7 \text{ Hz} \leq f_p \leq 2.4 \text{ Hz} \quad (1.30)$$

where f_p is the walking frequency, Hz, which is taken as the lowest harmonic of the fundamental frequency of the floor. The damping ratio, ζ , is taken as 1.1 % for bare floors and 3.0 % for normal, well furnished floors. The resonance build-up factor may conservatively be taken as 1.0 when the building parameters are unknown.

The effective vibrating modal mass, M , which accounts for the effective plan area participating in the motion, is calculated by

$$M = \frac{m L_{eff} S}{4} \quad (1.31)$$

where m = total floor distributed mass, kg/m². Table 1.1 provides equations to estimate the floor beam effective span, L_{eff} , and the floor effective width, S , for structural systems.

From Table 1.1, the relative flexibility of the primary beam, RF_{main_beam} , is calculated by the following expression

$$RF_{main_beam} = \frac{\Delta_g}{\Delta_g + \Delta_b} \quad (1.32)$$

where Δ_g and Δ_b are as previously described. The effective width of the floor participating in the motion, S^* , is measured in meters and is given by

$$S^* = 4.5 \left(\frac{EI_1}{m f_o^2} \right)^{1/4} \quad (1.33)$$

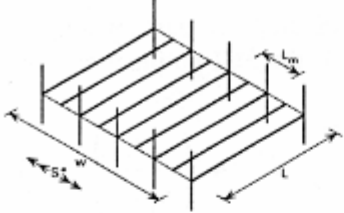
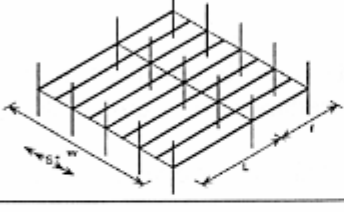
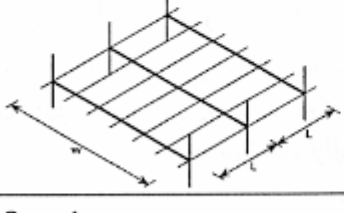
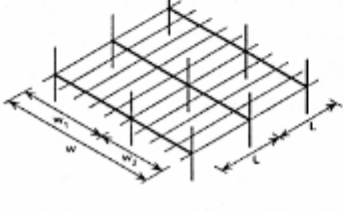
where EI_1 = dynamic flexural rigidity of the slab, Nm² per meter width; f_o = fundamental frequency of the floor, Hz. The effective span of the secondary beam participating in the motion, L^* , is measured in meters and is given by

$$L^* = 3.8 \left(\frac{EI_b}{m b f_o^2} \right)^{1/4} \quad (1.34)$$

where EI_b = dynamic flexural rigidity of the composite secondary beam, Nm²; and b = the secondary beam spacing, m. Table 1.1 also defines W as the width of the floor area being analyzed, L_m as the span of the primary beam measured in meters, and L_{max} as the

entire length of the secondary beam when considered to act continuously. These three parameters are measured in meters.

Table 1.1 – L_{eff} and S Dimensions (Hicks, et al. 2000)

Indicative floor layout		Qualifying conditions	L_{eff} (m)	S (m)
Mode shape governed by motion of secondary beams	Case 1 	$RF_{main\ beam} < 0.2$	L	S^* but $\leq W$
		$RF_{main\ beam} > 0.2$	L	Greater of S^* or L_m but $\leq W$
	Case 2 	$I = L$	$2L$	As for Case (1) above
		$0.8L < I < L$	$1.7L$	
		$I < 0.8L$	L	
Mode shape governed by motion of primary beams	Case 3 	$RF_{main\ beam} < 0.6$	$2L$	W
		$RF_{main\ beam} > 0.6$	L^* but $\leq L_{max}$	
	Case 4 	$W_2 = W_1$	As for Case (3) above	$2W_1$
		$W_2 > 0.8W_1$		$1.7W_1$
		$W_2 < 0.8W_1$		W_1

1.4.4 ADDITIONAL CURRENT RESEARCH

In recent years, floor vibrations research has expanded to study a multitude of related fields. By investigating a broad range of correlated topics, further understanding of the science of human induced floor vibrations can continue to evolve. Through the work of Kitterman (1994), the motion of steel joists and joist-girders was investigated. Specifically, the effective moment of inertia and the ability of joist seats to provide

composite behavior were explored. Kitterman derived equations for estimating the effective moment of inertia of framing members as a result of many tests of floor systems. Rottmann (1996) studied the use of tuned mass dampers for retrofitting floor systems with motion problems. The use of finite element analyses for predicting floor behavior was conducted by Band (1996), Beavers (1998), and Sladki (1999). Finite element analyses proved to be an effective tool in approximating the fundamental natural frequency of a floor. However, the studies also found that peak accelerations of floors cannot be predicted accurately using finite element techniques. Warmoth (2002) also investigated the girder moment of inertia in relation to floor vibrations. Warmoth recognized that different joist-seat connections vary in stiffness and proposed a new calculation for girder effective moment of inertia based on these findings. Through the work of Jackson (2002), floor motion properties of castellated beams were examined. Castellated beams have web openings creating non-prismatic cross sections. Jackson concluded that the composite moment of inertia should be calculated using the net moment of inertia for floor member properties.

1.5 NEED FOR RESEARCH

The dynamics of floors systems are changing with currents trends in design using lightweight, high-strength materials, and office fit-outs. Properties and vibrations behavior change due to less damping, stiffness, and mass provided by the floors. Therefore, past assumptions and methodology become out dated and floor vibrations research must continue to evolve. Analyses of existing floors are important for investigating more accurate techniques to improve the correlation between predicted and observed floor behavior. Examining case studies of floor systems can provide insight on the accuracy of the various prediction criteria. The focus of this research is divided into three topics in which case studies of existing floors are evaluated. In Chapter II, the fundamental natural frequencies of floor systems with various framing are studied. The measured and predicted bay, beam, and girder frequencies are scrutinized. Four acceptability criteria for analyzing existing occupied floors in the United States are compared in Chapter III. The criteria are then evaluated with field subjective analyses of the floors. The AISC and SCI methods on predicting floor motion are compared in

Chapter IV. Conclusions are made and continued research is proposed in Chapter V based on the findings from each study. Supplemental material in the publication by Boice and Murray (2003) for this report contains summary sheets of the case studies used in the frequency and criteria comparisons. These sheets include floor systems properties such as damping, framing, loading, predicted and measured fundamental frequencies, accelerations, and slab properties. In addition, framing plans for the floor systems evaluated in this study are found in the supplemental material. Appendix A contains the example calculations of frequencies and acceptability criteria based on the AISC and SCI methods.

CHAPTER II – NATURAL FREQUENCY STUDY

2.1 INTRODUCTION

Analyzing actual floor systems that are occupied or under construction provides insight into the behavior of vibrations induced by walking excitations. The fundamental natural frequency is an important property needed to predict the response of a floor. Therefore, the natural frequencies for floor systems of various framing are examined in this chapter. Correlation between predicted and measured response can be improved by comparing structural system frequencies.

This chapter provides a comparison of predicted and measured frequencies from 103 case studies involving floors occupied and or currently being constructed in the United States and the United Kingdom. The type of framing used differentiates the structural floor systems. The classifications for framing are “hot-rolled beams”, “hot-rolled girders with joists”, “hot-rolled girders with castellated beams”, and “joist girders with joists”. The controlling girder, which allows the most deflection corresponding to the lower frequency, is considered when classifying a floor system when two different types of girders are used in the framing.

The measured frequency was determined from the response of a floor system due to heel drops, walking, and ambient conditions. For ambient measurements, the frequency is measured without any excitation to the floor. An ONO SOKKI CF-1200 Handheld FFT Analyzer and seismic accelerometer was primarily used in the most recent case studies to measure natural frequencies as a result of excitations administered at the center of selected bays. This FFT analyzer is accurate to within 0.25 Hz.

The framing systems were modeled using the design software FLOORVIB2 (Murray and Elhouar 1994) in order to predict the fundamental natural frequency. The FLOORVIB2 software is based on the AISC Design Guide. In addition, the frequencies are also predicted using the methods outlined in the SCI Publication *Design Guide on the*

Vibration of Floors, (Wyatt 1989). The predicted beam, girder, and bay frequencies are then compared to the measured frequencies.

Supplemental material for this report is found in the publication by Boice and Murray (2003) and contains summary sheets of the case studies used in the frequency and criteria comparisons. These sheets include floor systems properties such as damping, framing, loading, predicted and measured fundamental frequencies, accelerations, and slab properties. In addition, the supplemental material contains framing plans for the floor systems evaluated in this chapter. Appendix A contains sample calculations of beam, girder, and bay frequencies. The following sections of this chapter include the frequency comparison using the Design Guide and SCI methods, followed by a summary of AISC and SCI predicted bay frequencies. In addition, this chapter includes vibration properties for floors that were tested in Europe. The final section of this chapter provides conclusions and recommendations.

2.2 AISC FREQUENCY COMPARISONS

This section presents the results from comparisons between measured bay frequencies and predicted beam, joist, girder, and system frequencies. The predicted frequencies are calculated using the methods outlined in the AISC Design Guide. The floors are classified and compared by the framing used in the structural system. The bay or system frequency is predicted using Dunkerly's relationship, Equation 1.12. In addition, system frequencies estimated using modifications to Dunkerly's equation are compared with measured frequencies. These modifications raise the powers of the system properties to 4 and 6. The following relationships reflect the revisions,

$$\frac{1}{f_n^4} = \frac{1}{f_j^4} + \frac{1}{f_g^4} \quad (2.1)$$

$$\frac{1}{f_n^6} = \frac{1}{f_j^6} + \frac{1}{f_g^6} \quad (2.2)$$

where f_n = system frequency, f_j = beam or joist frequency, f_g = girder frequency, measured in Hz.

In the following sections, Tables 2.1 through 2.4 provide predicted and measured frequencies as well as frequency ratios for each type of framing. The girders are assumed

to be infinitely rigid for some of the case studies. Therefore, the girders that do not participate in the motion of the floor are modeled as walls in the FLOORVIB2 software. Tables 2.1 through 2.4 have the phrase “N/A” or not applicable when the girder frequency is not used in the comparison. In Tables 2.1 through 2.4, measured bay frequencies are compared with predicted bay frequencies. In the comparisons that follow, f_n from equations 1.12, 2.1, and 2.2 will be referred to as f_n^2 , f_n^4 , and f_n^6 , respectively. In addition, measured system frequencies are compared with predicted beam and girder frequencies in these tables. Figures 2.1 through 2.5 in the following sections graphically show measured bay frequencies versus predicted frequencies. In these figures, the abbreviation “H-R” describes hot-rolled framing, “H-R/J” describes framing with hot-rolled girders and joists, “J-G/J” describes framing with joist girders and joists, and “H-R/Cast.” describes framing with hot-rolled girders and castellated beams. Also, in Figures 2.1 through 2.5, the solid line represents when the predicted frequency is equal to the measured frequency. The dashed lines located above and below the solid line represent a range of predicted frequencies within 10 % of the measured frequencies.

2.2.1 FLOORS WITH HOT-ROLLED SHAPES

Table 2.1 summarizes the results from the frequency comparisons for floors that are constructed of hot-rolled framing. From Table 2.1, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.03 with a modest standard deviation of 0.18. The ratio comparing predicted girder and measured frequencies demonstrates that the girder does not solely control the floor behavior. The f_g/f_m ratio is 1.34 with a standard deviation of 0.48. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.80 with a standard deviation of 0.13. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , provides better correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 0.90 with a standard deviation of 0.15. The average f_n^6/f_m ratio is 0.93 with a standard deviation of 0.16.

The measured bay frequencies are plotted against the predicted beam frequencies in Figure 2.1(a). This figure illustrates that the predicted beam frequencies are within 10 % of the measured bay frequencies for about 64 % of the case studies with hot-rolled

framing. The measured bay frequencies are plotted against the predicted bay frequencies, f_n^2 , in Figure 2.1(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 28 % of the case studies with hot-rolled framing. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.1(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for 40 % of the case studies with hot-rolled framing. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.1(d). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 28 % of the case studies with hot-rolled framing.

Table 2.1 – AISC Frequency Comparison for Floors with Hot-Rolled Shapes

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Jersey Lils	272	Typ.	3.23	10.40	3.15	3.22	3.23	4.93	0.655	2.110	0.639	0.654	0.655
Mercy Health Ctr.	508	Int. Ext.	9.97	7.19	6.09	6.77	7.03	8.88	1.123	0.810	0.686	0.763	0.792
			8.92	7.58	6.09	6.83	7.19	8.88	1.005	0.854	0.686	0.769	0.809
Iowa State	535	1	4.07	5.15	3.19	3.75	3.92	3.88	1.049	1.327	0.822	0.966	1.012
		2,3	4.12	9.11	3.82	4.08	4.11	3.75	1.099	2.429	1.019	1.087	1.097
		4	4.12	9.13	3.82	4.08	4.11	4.00	1.030	2.283	0.955	1.020	1.029
Oracle Building	565	1,2	4.87	5.83	3.81	4.41	4.64	5.25	0.928	1.110	0.726	0.840	0.883
		3,4	4.87	5.83	3.81	4.41	4.64	4.13	1.179	1.412	0.923	1.068	1.123
		5	5.15	5.69	3.88	4.53	4.79	5.50	0.936	1.035	0.705	0.824	0.870
Newgen Building	568	Ext.	6.62	9.53	5.47	6.28	6.50	6.50	1.018	1.466	0.842	0.967	1.001
42nd St. Building	575	Typ.	6.50	3.95	3.38	3.83	3.92	4.06	1.601	0.973	0.833	0.942	0.965
Penn State	Boice3	Typ.	4.51	8.66	4.23	4.43	4.50	4.90	0.920	1.767	0.863	0.904	0.917
Newport V	600	Typ.	3.78	6.80	3.38	3.69	3.76	4.00	0.945	1.700	0.845	0.924	0.940
Doylestown Hosp.	569	Typ.	6.86	6.98	5.19	5.82	6.16	7.00	0.980	0.997	0.741	0.831	0.880
UT Med. Center	Boice1	C	13.47	11.96	8.94	10.60	11.19	13.00	1.036	0.920	0.688	0.815	0.861
		D	12.73	7.53	6.48	7.32	7.48	13.50	0.943	0.558	0.480	0.542	0.554
One Texas Court	617	1	4.83	6.29	3.96	4.48	4.68	5.25	0.920	1.198	0.754	0.854	0.892
		2	6.54	6.36	4.61	5.42	5.74	6.50	1.006	0.978	0.709	0.834	0.883
AMEX	602	Typ.	4.24	N/A	4.24	4.24	4.24	5.25	0.808	N/A	0.808	0.808	0.808
GTE	Slad2	Typ.	5.06	6.17	4.00	4.61	4.84	5.50	0.920	1.122	0.727	0.838	0.880
Three Parkwood	618	1	5.35	6.79	4.20	4.93	5.16	4.25	1.259	1.598	0.988	1.160	1.215
		2	4.62	5.77	3.65	4.24	4.44	4.00	1.155	1.443	0.913	1.060	1.111
Norman Pointe 1	620	1	6.05	7.97	4.91	5.63	5.88	5.00	1.210	1.594	0.982	1.126	1.175
		3	5.12	4.53	3.39	4.02	4.24	5.25	0.975	0.863	0.646	0.766	0.808
		4	6.05	7.97	4.91	5.63	5.88	5.25	1.152	1.518	0.935	1.073	1.119

Average -	1.034	1.336	0.797	0.897	0.931
Standard Deviation -	0.176	0.480	0.132	0.151	0.158

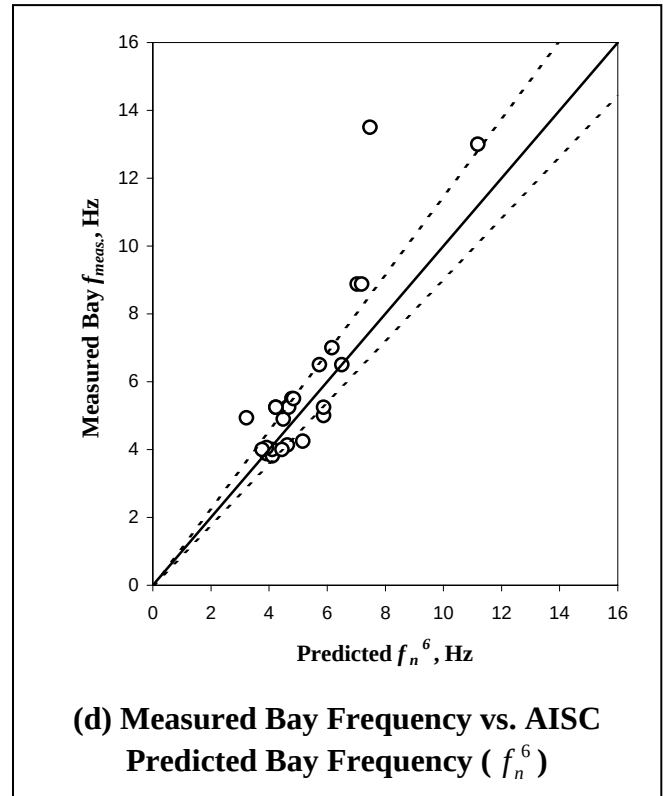
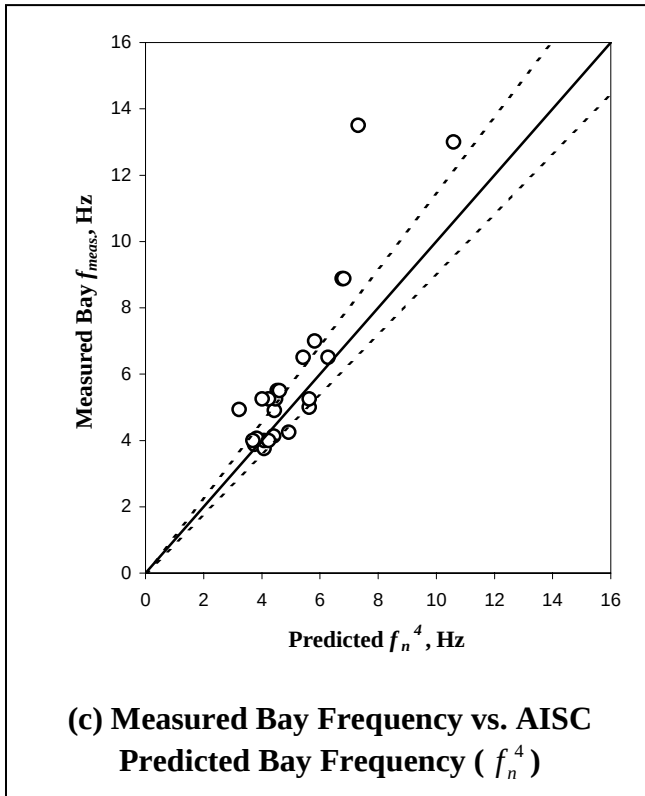
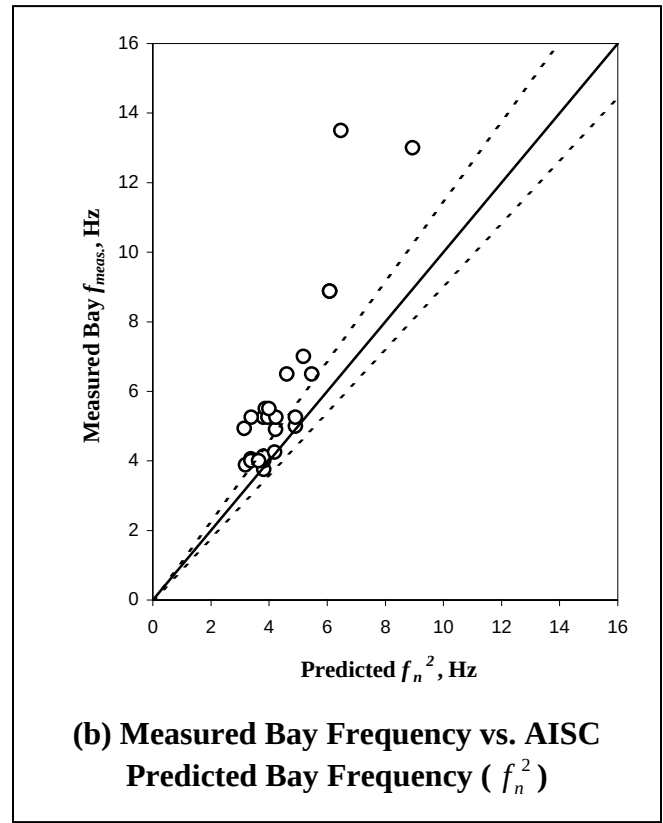
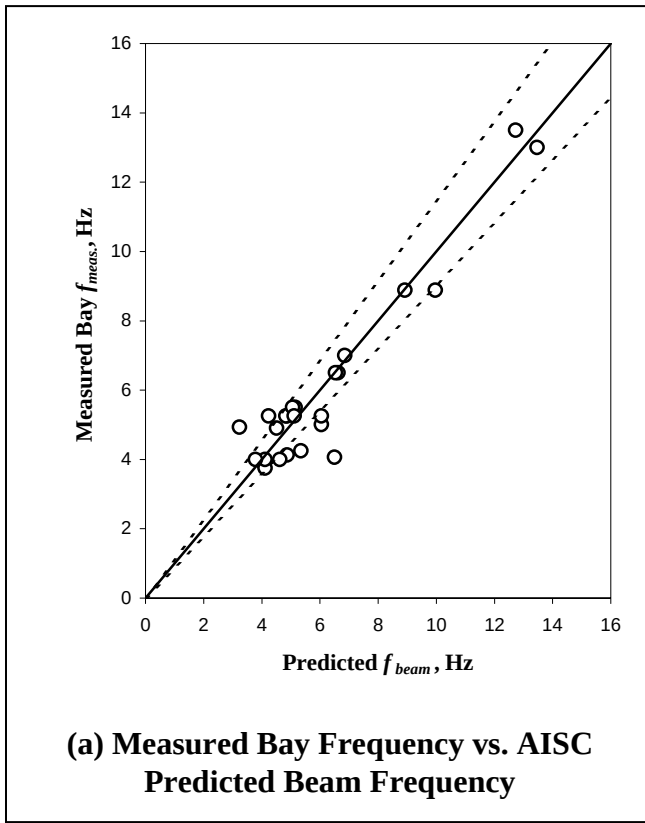


Figure 2.1 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Shapes

2.2.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS

Table 2.2 summarizes the results from the frequency comparison for floors that are constructed of hot-rolled girders and joist framing. From Table 2.2, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.06 with a standard deviation of 0.25. The f_g/f_m ratio is 1.67 with a high standard deviation of 0.60. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.90 with a standard deviation of 0.20. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 0.98 with a standard deviation of 0.21. The average f_n^6/f_m ratio is 1.00 with a standard deviation of 0.22.

Table 2.2 – AISC Frequency Comparison for Hot-Rolled Girders and Joists

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Premiere	351	1	10.67	7.08	5.90	6.77	6.98	6.00	1.778	1.180	0.983	1.129	1.164
Patterson Ave. Fac.	549	4	6.58	12.57	5.97	6.46	6.56	4.50	1.462	2.793	1.327	1.436	1.457
		5	6.58	10.41	5.75	6.34	6.51	4.50	1.462	2.313	1.278	1.409	1.447
		7	5.51	7.51	4.68	5.17	5.38	4.50	1.224	1.669	1.040	1.149	1.195
Orange Synagogue	349	1	7.10	7.57	5.36	6.15	6.51	7.50	0.947	1.009	0.715	0.820	0.868
		4	7.10	7.57	5.36	6.15	6.51	8.50	0.835	0.891	0.631	0.724	0.766
		5	7.10	7.57	5.36	6.15	6.51	8.00	0.888	0.946	0.670	0.769	0.814
ACTIC	341	Typ.	6.21	12.40	5.64	6.12	6.19	6.50	0.955	1.908	0.868	0.941	0.953
Camron Univ.	514	271	6.16	10.98	5.65	6.02	6.13	7.00	0.880	1.569	0.807	0.859	0.876
		273	5.88	10.50	5.39	5.74	5.85	6.75	0.871	1.556	0.799	0.851	0.867
Melroe	508	Typ.	5.46	5.28	3.80	4.51	4.78	5.50	0.993	0.960	0.691	0.820	0.869
Steel Case	570	Mezz.	5.25	4.92	3.59	4.26	4.51	4.75	1.105	1.036	0.756	0.898	0.950
		Int.	5.25	4.02	3.19	3.73	3.90	4.75	1.105	0.846	0.672	0.786	0.821
UT Med.	Boice1	A	5.23	9.29	4.76	5.11	5.20	5.00	1.046	1.858	0.952	1.021	1.041
		B	5.22	9.16	4.73	5.22	5.22	4.75	1.099	1.928	0.996	1.099	1.099
IQ/ Ford	615	Typ.	5.31	5.96	3.97	4.70	4.96	4.75	1.118	1.255	0.836	0.989	1.045
XILINX	609	Typ.	5.05	6.06	3.93	4.58	4.81	4.75	1.063	1.276	0.827	0.964	1.013
CO Civic Center	610	1	5.16	N/A	5.16	5.16	5.16	5.25	0.983	N/A	0.983	0.983	0.983
		2	5.26	N/A	5.26	5.26	5.26	5.25	1.002	N/A	1.002	1.002	1.002
		3	4.70	N/A	4.70	4.70	4.70	5.50	0.855	N/A	0.855	0.855	0.855
Gateway	Boice2	Typ.	2.52	4.07	2.22	2.44	2.50	4.00	0.630	1.018	0.555	0.609	0.624
Ruby Tuesday	Slad1	2nd.	4.49	10.50	4.26	4.45	4.49	4.25	1.056	2.471	1.002	1.048	1.055
		3rd.	3.86	10.56	3.72	3.84	3.86	3.75	1.029	2.816	0.992	1.025	1.029
Midwest Kentworth	Band3	Typ.	4.04	N/A	4.04	4.04	4.04	5.75	0.703	N/A	0.703	0.703	0.703
Nations Bank	Band4	1,4	4.04	8.09	3.77	3.98	4.03	3.85	1.049	2.101	0.979	1.034	1.047
		2,6	3.53	8.09	3.36	3.50	3.53	5.00	0.706	1.618	0.672	0.700	0.705
		3	3.53	8.09	3.36	3.50	3.53	3.90	0.905	2.074	0.862	0.897	0.904
		5	4.04	8.09	3.77	3.98	4.03	3.40	1.188	2.379	1.109	1.170	1.185
Eight Parkwood	618	2	7.61	10.82	6.22	7.20	7.47	5.25	1.450	2.061	1.185	1.372	1.422
		6	7.61	10.82	6.22	7.20	7.47	5.50	1.384	1.967	1.131	1.310	1.358

Average -	1.059	1.673	0.896	0.979	1.004
Standard Deviation -	0.252	0.602	0.195	0.214	0.218

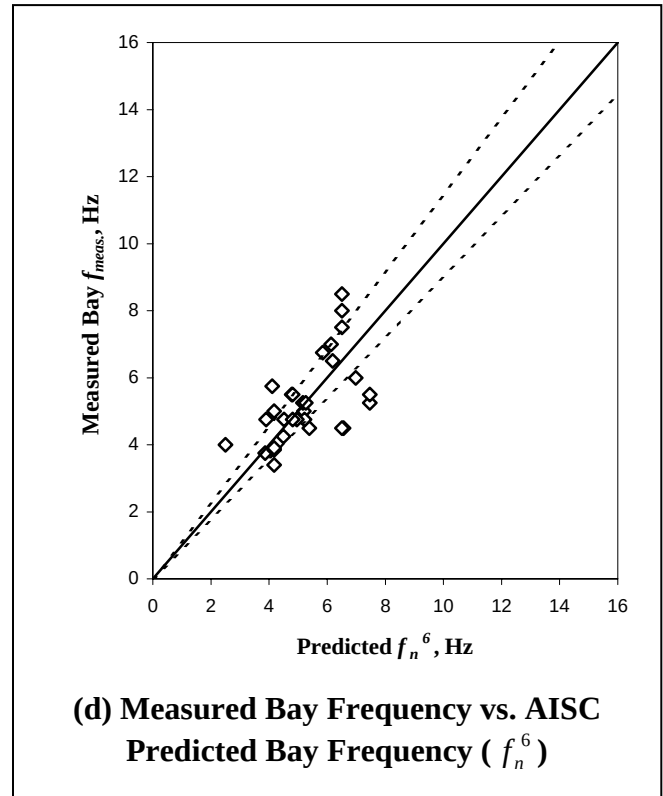
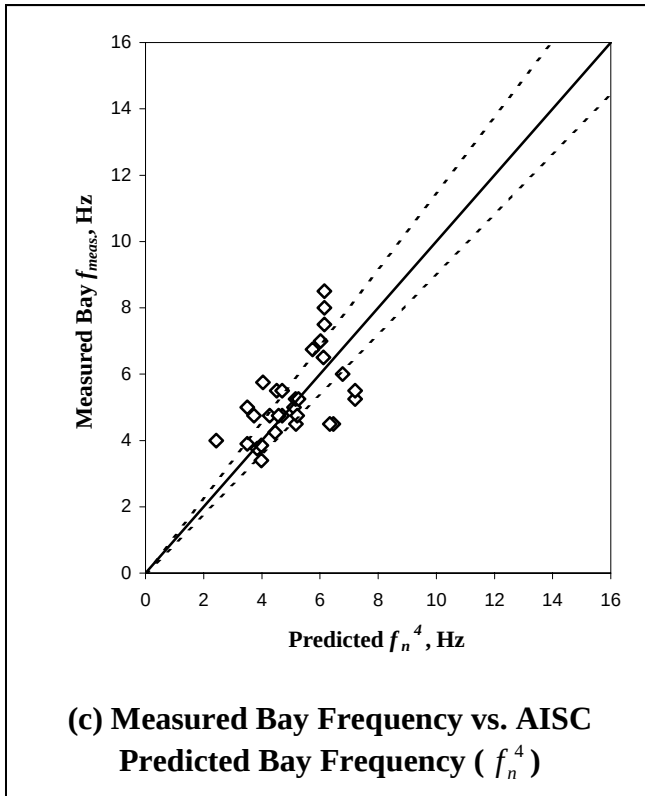
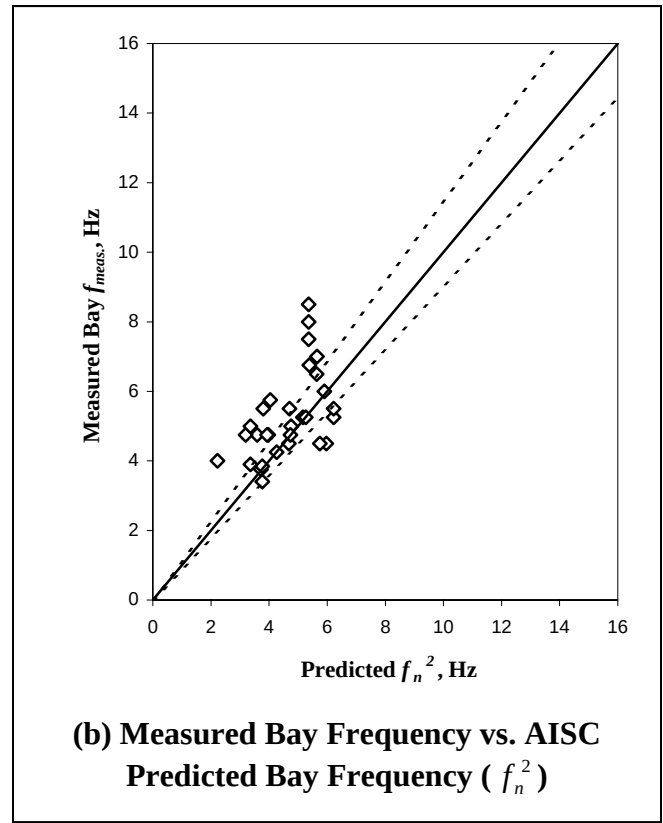
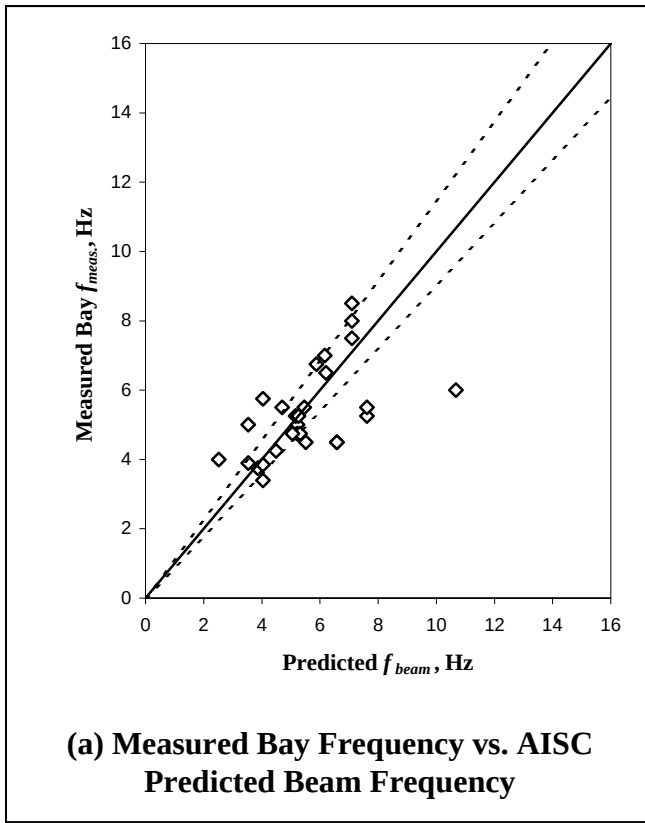


Figure 2.2 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Girders and Joists

The measured bay frequencies are plotted against the predicted beam frequencies in Figure 2.2(a). This figure illustrates that the predicted beam frequencies are within 10 % of the measured bay frequencies for only about 40 % of the case studies with hot-rolled girders and joist framing. The measured bay frequencies are plotted against the predicted bay frequencies, f_n^2 , in Figure 2.2(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 30 % of the case studies with hot-rolled girders and joist framing. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.2(c). This figure reveals that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 33 % of the case studies with hot-rolled girders and joist framing. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.2(d). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 40 % of the case studies with hot-rolled girders and joist framing.

2.2.3 FLOORS WITH JOIST GIRDERS AND JOISTS

Table 2.3 summarizes the results from the frequency comparison for floors that are constructed using joist girders and joist framing. From Table 2.3, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.00 with a modest standard deviation of 0.23. The ratio comparing predicted girder and measured frequencies reveals that the primary framing may control the floor behavior for some case studies using this particular framing. The f_g/f_m ratio is 1.00 with a standard deviation of 0.24. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.76 with a standard deviation of 0.17. The comparison between measured and predicted system frequencies, f_n^4 and f_n^6 , provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 0.84 with a standard deviation of 0.17. The average f_n^6/f_m ratio is 0.87 with a standard deviation of 0.18.

The measured bay frequencies are plotted against the predicted beam frequencies in Figure 2.3(a). This figure illustrates that the predicted beam frequencies are within 10 % of the measured bay frequencies for only about 39 % of the case studies using joist girders and joists for framing. The measured bay frequencies are plotted against the

predicted bay frequencies, f_n^2 , in Figure 2.3(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 18 % of the case studies using joist girders and joists for framing. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.3(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 29 % of the case studies using joist girders and joists for framing. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.3(d). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 32 % of the case studies using joist girders and joists for framing.

Table 2.3 – AISC Frequency Comparison for Floors with Joist Girders and Joists

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Z.C.M.I. Store	269	1--4	7.23	6.22	4.72	5.58	5.88	5.65	1.280	1.101	0.835	0.987	1.040
		5,10	6.86	4.99	4.03	4.69	4.88	4.75	1.444	1.051	0.848	0.988	1.027
		9	7.62	6.63	5.00	5.92	6.24	7.04	1.082	0.942	0.710	0.841	0.887
		7,8,11	7.23	6.22	4.72	5.58	5.88	5.11	1.415	1.217	0.924	1.091	1.150
Patterson	549	6	6.04	6.10	4.67	5.10	5.41	5.00	1.208	1.220	0.934	1.021	1.081
Toyota	481	Typ.	4.94	8.02	4.53	4.78	4.90	5.63	0.877	1.425	0.805	0.848	0.870
Garmin Int. Bldg.	506	Int.	6.25	4.52	3.66	4.25	4.42	6.25	1.000	0.723	0.586	0.681	0.707
		Ext.	6.25	4.52	3.66	4.25	4.42	6.25	1.000	0.723	0.586	0.681	0.707
Sony Corp.	511	Typ.	5.47	4.44	3.45	4.06	4.26	5.50	0.995	0.807	0.627	0.738	0.774
Diversa Building	568	Ext.-1	6.12	6.45	4.44	5.28	5.59	5.75	1.064	1.122	0.772	0.918	0.971
		Ext.-2	4.71	5.00	3.43	4.07	4.31	5.00	0.942	1.000	0.686	0.815	0.862
		Int.-1	9.52	7.29	5.79	6.77	7.07	9.50	1.002	0.767	0.609	0.713	0.744
		Int.-2	7.24	5.57	4.42	5.17	5.40	13.00	0.557	0.428	0.340	0.397	0.415
Pacific Tech.	568	Typ.	6.14	6.71	4.53	5.38	5.69	6.00	1.023	1.118	0.755	0.896	0.948
IBM	605	Typ.	6.78	N/A	6.78	6.78	6.78	6.00	1.130	N/A	1.130	1.130	1.130
Southern Bell	Band1	2	5.54	6.67	4.48	5.03	5.28	6.25	0.886	1.067	0.717	0.804	0.845
		5,7	5.54	6.67	4.48	5.03	5.28	6.75	0.821	0.988	0.664	0.745	0.783
		9	7.58	6.79	5.22	6.00	6.33	9.50	0.798	0.715	0.549	0.631	0.667
Wholesale Grocers	Band2	1	2.45	N/A	2.45	2.45	2.45	3.50	0.700	N/A	0.700	0.700	0.700
		3	2.53	N/A	2.53	2.53	2.53	4.25	0.595	N/A	0.595	0.595	0.595
		5	2.03	N/A	2.03	2.03	2.03	3.00	0.677	N/A	0.677	0.677	0.677
Eight Parkwood	618	1	6.11	5.89	4.24	5.04	5.34	5.25	1.164	1.122	0.808	0.960	1.017
		3	6.11	6.10	4.32	5.13	5.44	5.75	1.063	1.061	0.751	0.893	0.946
		4,5	6.11	5.89	4.24	5.04	5.34	4.50	1.358	1.309	0.942	1.120	1.186
R.S.I. Building	621	1	8.54	N/A	8.54	8.54	8.54	9.50	0.899	N/A	0.899	0.899	0.899
		2	8.54	N/A	8.54	8.54	8.54	9.75	0.876	N/A	0.876	0.876	0.876
		3	9.61	N/A	9.61	9.61	9.61	9.75	0.986	N/A	0.986	0.986	0.986
		4	6.87	N/A	6.87	6.87	6.87	7.25	0.948	N/A	0.948	0.948	0.948

Average -	1.003	0.995	0.759	0.842	0.873
Standard Deviation -	0.225	0.240	0.166	0.172	0.181

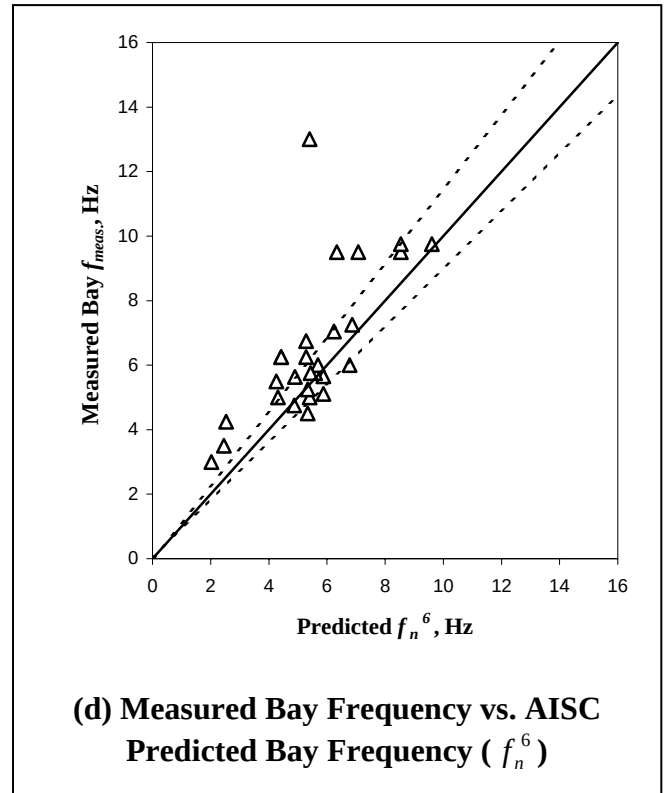
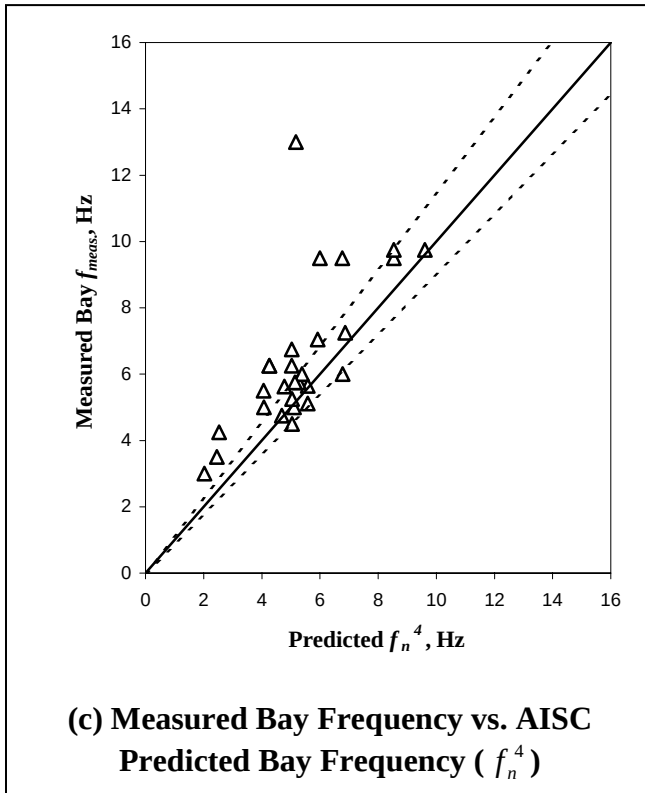
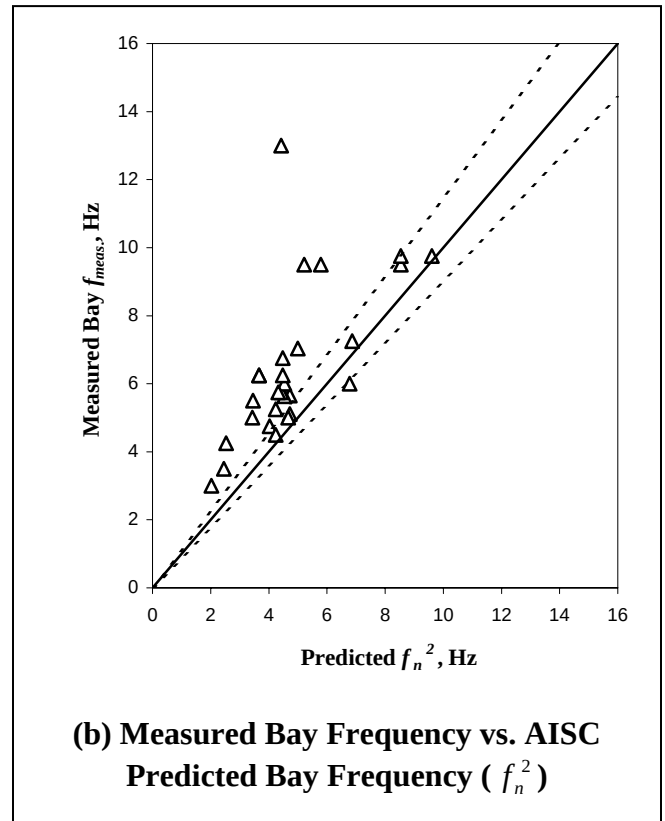
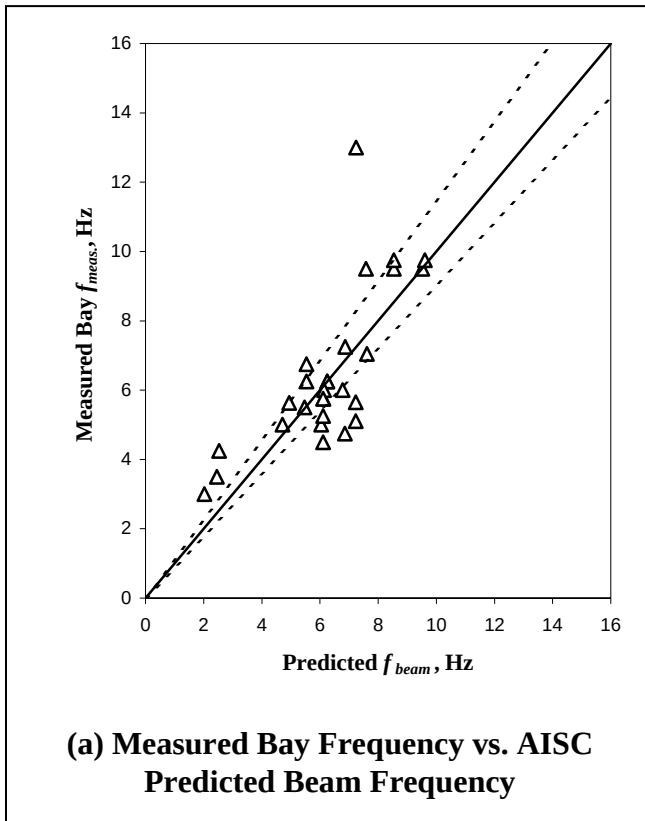


Figure 2.3 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Joist Girders and Joists

2.2.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS

Table 2.4 summarizes the results from the frequency comparison for floors that are constructed of hot-rolled girders with castellated beams. From Table 2.4, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.06 with a low standard deviation of 0.08. The ratio comparing predicted and measured girder frequencies reveals that the primary framing does not solely control floor behavior. The f_g/f_m ratio is 1.56 with a standard deviation of 0.54. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.86 with a low standard deviation of 0.09. The average f_n^4/f_m ratio is 0.97 with a standard deviation of 0.05. The average f_n^6/f_m ratio is 1.00 with a standard deviation of 0.05.

Table 2.4 – AISC Frequency Comparison for Floors with Hot-Rolled Girders and Castellated Beams

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Lennox	572	1,3	3.41	7.16	3.19	3.37	3.40	3.32	1.029	2.160	0.962	1.016	1.027
Wal-Mart	572	4,5	4.66	10.22	4.44	4.61	4.65	4.75	0.981	2.152	0.935	0.971	0.980
AMAT	Boice7	B,B2	6.93	7.88	5.21	6.16	6.50	6.75	1.027	1.167	0.772	0.913	0.964
		C	5.63	5.79	4.04	4.80	5.08	4.75	1.185	1.219	0.851	1.010	1.070
		D	6.93	7.21	5.00	5.94	6.29	6.50	1.066	1.109	0.769	0.914	0.968
Average -									1.058	1.561	0.858	0.965	1.002
Standard Deviation -									0.078	0.544	0.090	0.050	0.046

The measured bay frequencies are plotted against the predicted beam frequencies in Figure 2.4(a). This figure illustrates that the predicted beam frequencies are within 10 % of the measured bay frequencies for about 80 % of the case studies with hot-rolled girders and castellated framing. The measured bay frequencies are plotted against the predicted bay frequencies, f_n^2 , in Figure 2.4(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 40 % of case studies with hot-rolled girders and castellated beams. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.4(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for all of the case studies with hot-rolled girders and castellated beams. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.4(d). This figure illustrates that the predicted bay frequencies are

within 10 % of the measured bay frequencies for all of the case studies with hot-rolled girders and castellated framing.

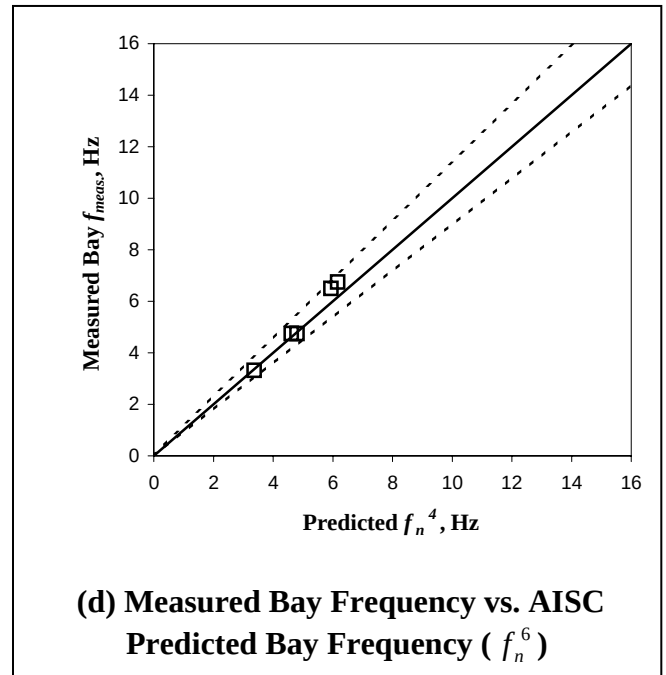
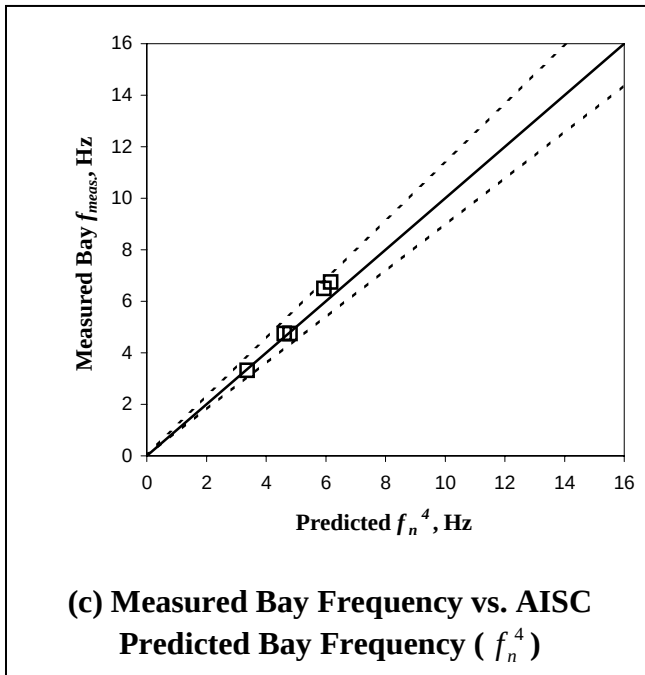
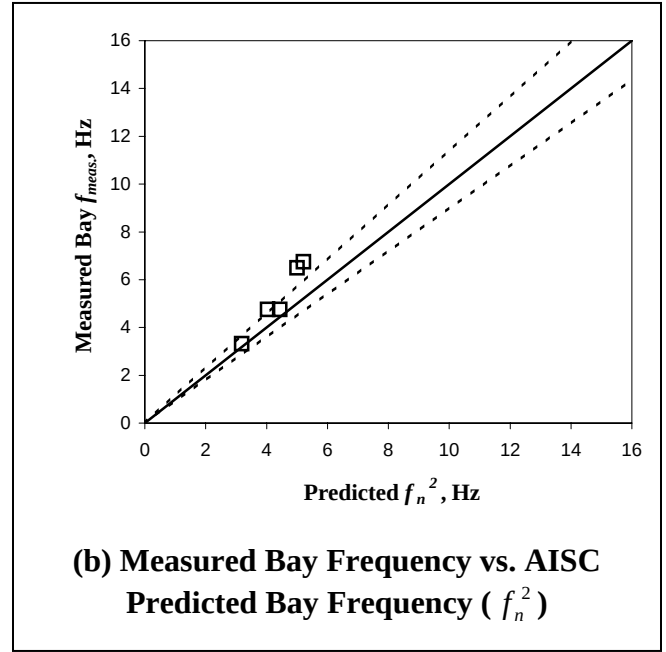
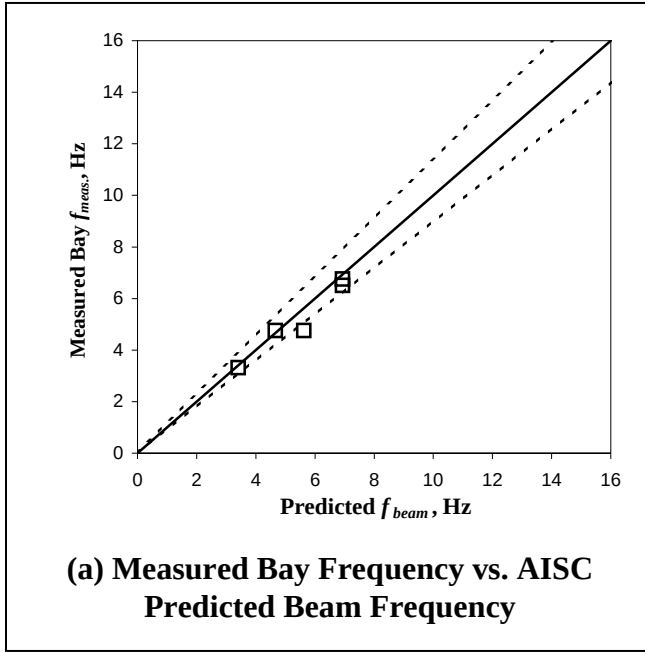


Figure 2.4 – Measured Bay Frequencies vs. AISC Predicted Frequencies for Floors with Hot-Rolled Girders with Castellated Beams

2.2.5 SUMMARY OF AISC FREQUENCY COMPARISONS

Based on a statistical analysis of the data in Tables 2.1 through 2.4, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.03 with a standard deviation of 0.22 for all of the floor systems used in these comparisons. The ratio comparing predicted and measured girder frequencies shows that the primary framing does not solely control floor behavior. The f_g/f_m ratio is 1.38 with a high standard deviation of 0.55. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.82 with a modest standard deviation of 0.17. The average f_n^4/f_m ratio is 0.91 with a standard deviation of 0.19. The average f_n^6/f_m ratio is 0.94 with a standard deviation of 0.19.

The measured bay frequencies are plotted against the predicted beam frequencies for all types of floor framing in Figure 2.5(a). This figure illustrates that the predicted beam frequencies are within 10 % of the measured bay frequencies for only about 48 % of the case studies. The measured bay frequencies are plotted against the predicted bay frequencies, f_n^2 , in Figure 2.5(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 27 % of case studies. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.5(c). This figure illustrates that the predicted system frequencies are within 10 % of the measured bay frequencies for about 37 % of the case studies. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.5(d). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 37 % of the case studies.

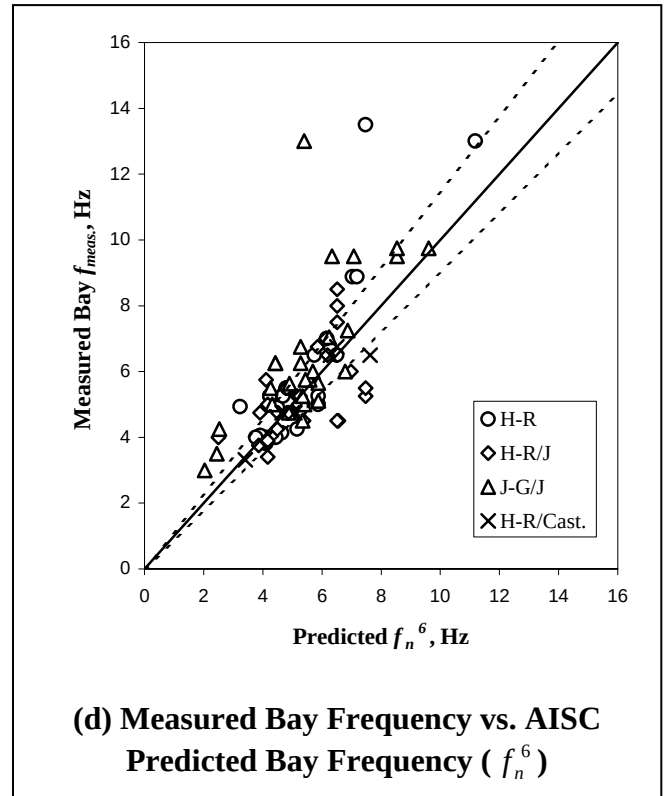
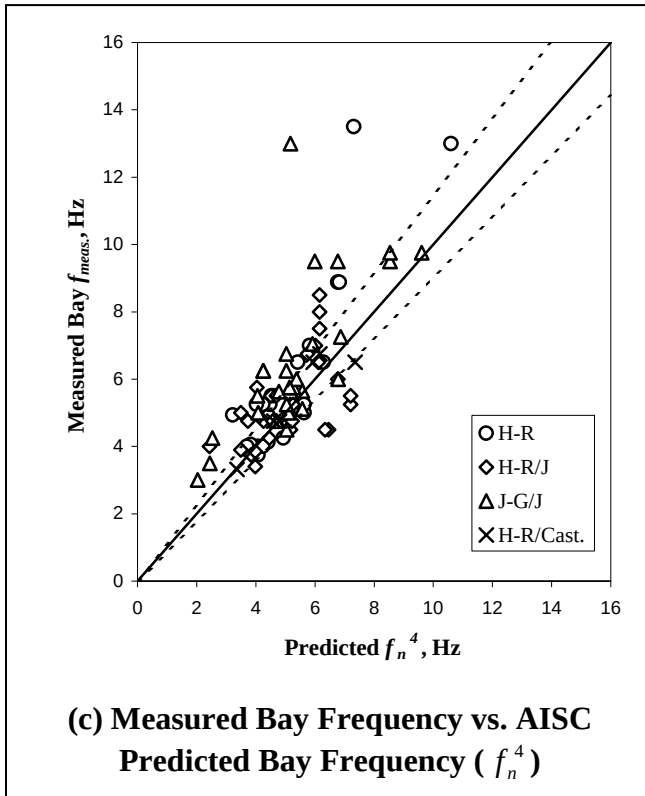
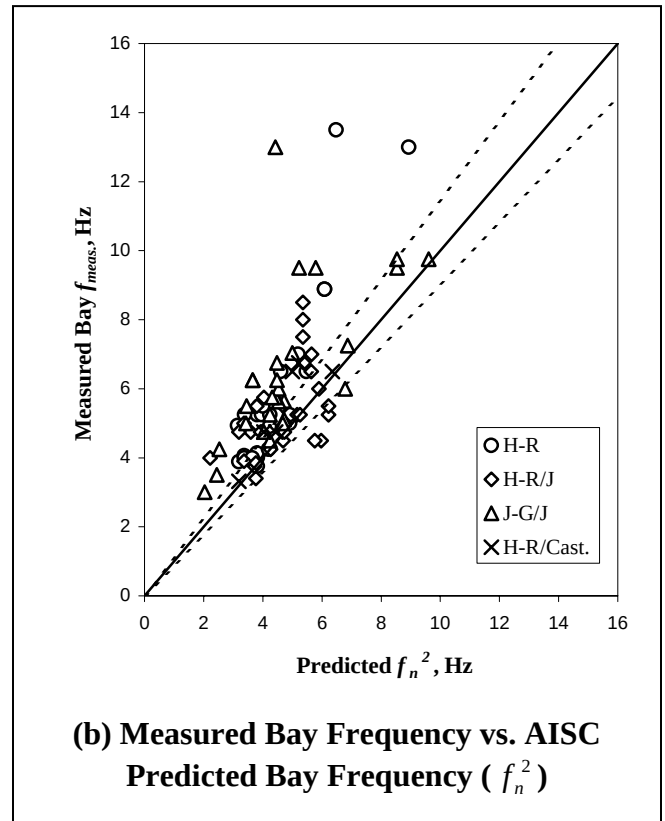
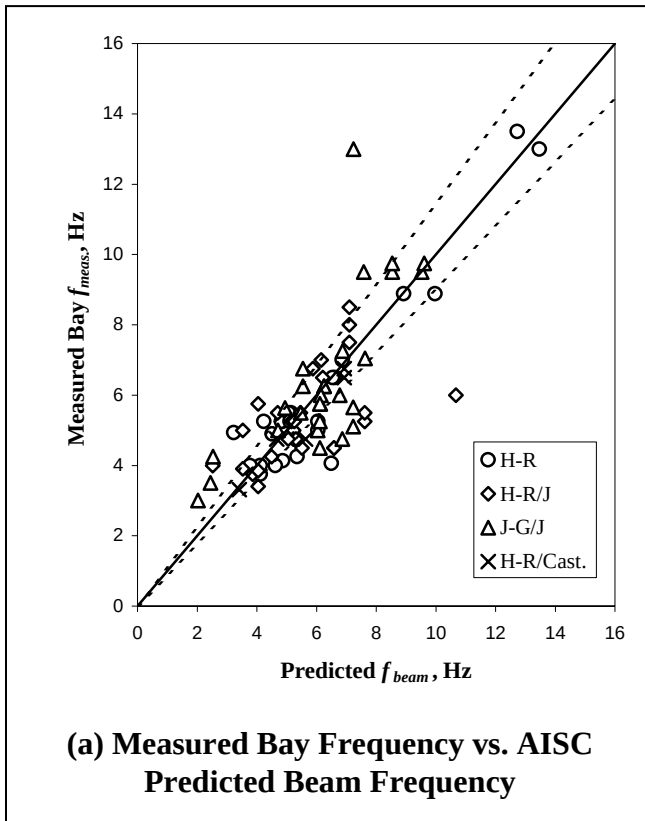


Figure 2.5 – Measured Bay Frequencies vs. AISC Predicted Frequencies for All Framing Types

2.3 SCI FREQUENCY COMPARISONS

This section presents the results from a comparison between measured bay frequencies and predicted beam, joist, girder, and system frequencies. The predicted frequencies are calculated using the methods outlined in the SCI Publication *Design Guide on the Vibration of Floors* (Wyatt 1989). The floors are classified and compared by the framing used in the structural system. The data for the SCI frequency comparisons is reported as was done in the previous section. In the following sections, Tables 2.5 through 2.8 provide predicted and measured frequencies as well as frequency ratios for each type of framing. Figures 2.6 through 2.10 in the following sections graphically show measured bay frequencies versus predicted frequencies. Measured bay frequencies are not plotted versus predicted beam frequencies in this section due to poor correlation.

2.3.1 FLOORS WITH HOT-ROLLED SHAPES

Table 2.5 summarizes the results from the frequency comparisons for floors that are constructed of hot-rolled framing. According to SCI predictions, the ratio comparing predicted girder and measured frequencies demonstrates that the beam does not solely control the floor behavior. From Table 2.5, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.62 with a high standard deviation of 0.73. The average ratio between predicted girder and measured bay frequencies (f_g/f_m) is 0.93 with a standard deviation of 0.18. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.96 with a standard deviation of 0.19. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , marginally provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 0.99 with a standard deviation of 0.19. The average f_n^6/f_m ratio is 1.00 with a standard deviation of 0.19.

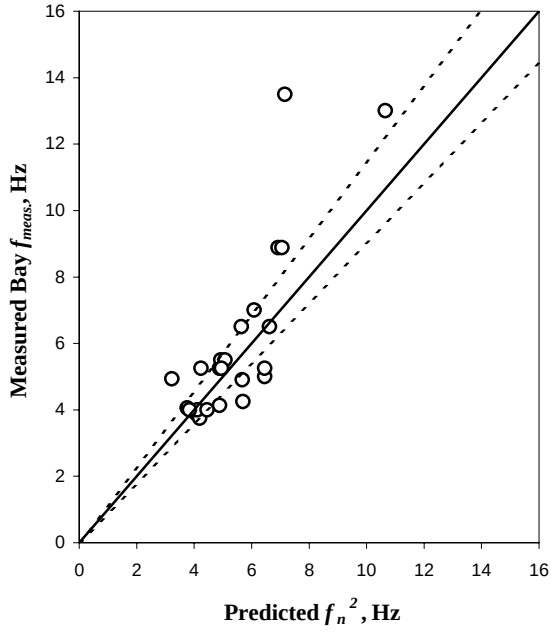
The measured bay frequencies are plotted versus predicted bay frequencies, f_n^2 , in Figure 2.6(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 32 % of the case studies with hot-rolled framing. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.6(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 48 % of the case studies with hot-rolled framing. The measured frequencies are plotted against the

predicted bay frequencies, using the revision f_n^6 , in Figure 2.6(c). This figure also illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 48 % of the case studies with hot-rolled framing.

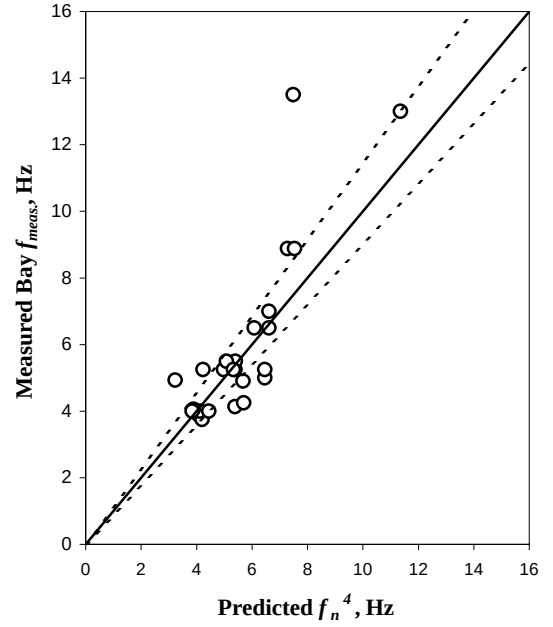
Table 2.5 – SCI Frequency Comparison for Floors with Hot-Rolled Shapes

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Jersey Lils Club	272	Typ.	3.23	N/A	3.23	3.23	3.23	4.93	0.655	N/A	0.655	0.655	0.655
Mercy Health Center	508	Int.	21.58	7.31	6.92	7.29	7.31	8.88	2.430	0.823	0.779	0.821	0.823
		Ext.	19.44	7.58	7.06	7.54	7.58	8.88	2.189	0.854	0.795	0.849	0.853
Iowa State University	535	1	4.07	N/A	4.07	4.07	4.07	3.88	1.049	N/A	1.049	1.049	1.049
		2,3	4.20	N/A	4.20	4.20	4.20	3.75	1.120	N/A	1.120	1.120	1.120
		4	4.12	N/A	4.12	4.12	4.12	4.00	1.030	N/A	1.030	1.030	1.030
Oracle Building	565	1,2	10.89	5.47	4.89	5.39	5.46	5.25	2.074	1.042	0.931	1.026	1.039
		3,4	10.89	5.47	4.89	5.39	5.46	4.13	2.637	1.324	1.184	1.304	1.321
		5	11.46	5.47	4.93	5.40	5.46	5.50	2.084	0.995	0.896	0.982	0.993
Newgen Building	568	Ext.	6.62	N/A	6.62	6.62	6.62	6.50	1.018	N/A	1.018	1.018	1.018
42nd St. Building	575	Typ.	14.37	3.90	3.76	3.89	3.90	4.06	3.539	0.961	0.926	0.959	0.961
Penn State	Boice3	Typ.	5.68	N/A	5.68	5.68	5.68	4.90	1.159	N/A	1.159	1.159	1.159
Newport V Building	600	Typ.	3.84	N/A	3.84	3.84	3.84	4.00	0.960	N/A	0.960	0.960	0.960
Doylestown Hospital	569	Typ.	14.94	6.67	6.09	6.61	6.66	7.00	2.134	0.953	0.870	0.944	0.952
UT Medical Center	Boice1	C	29.33	11.43	10.65	11.37	11.42	13.00	2.256	0.879	0.819	0.874	0.879
		D	28.40	7.50	7.16	7.49	7.50	13.50	2.104	0.556	0.530	0.555	0.556
One Texas Court	617	1	4.97	N/A	4.97	4.97	4.97	5.25	0.947	N/A	0.947	0.947	0.947
		2	14.33	6.14	5.64	6.09	6.13	6.50	2.205	0.945	0.868	0.937	0.944
AMEX Building	602	Typ.	4.24	N/A	4.24	4.24	4.24	5.25	0.808	N/A	0.808	0.808	0.808
GTE Building	Slad2	Typ.	5.08	N/A	5.08	5.08	5.08	5.50	0.924	N/A	0.924	0.924	0.924
Three Parkwood Building	618	1	5.70	N/A	5.70	5.70	5.70	4.25	1.341	N/A	1.341	1.341	1.341
		2	4.45	N/A	4.45	4.45	4.45	4.00	1.113	N/A	1.113	1.113	1.113
Norman Pointe 1	620	1	6.46	N/A	6.46	6.46	6.46	5.00	1.292	N/A	1.292	1.292	1.292
		3	12.57	5.38	4.95	5.34	5.37	5.25	2.394	1.025	0.943	1.016	1.024
		4	6.46	N/A	6.46	6.46	6.46	5.25	1.230	N/A	1.230	1.230	1.230

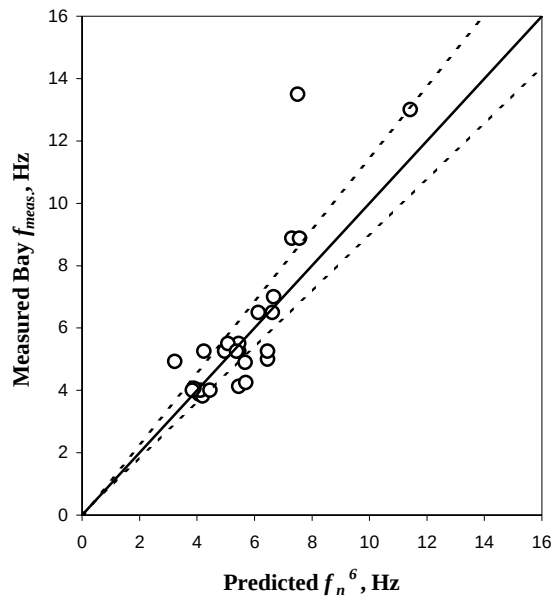
Average -	1.617	0.931	0.963	0.992	0.995
Standard Deviation -	0.727	0.183	0.194	0.189	0.189



**(a) Measured Bay Frequency vs. SCI
Predicted Bay Frequency (f_n^2)**



**(b) Measured Bay Frequency vs. SCI
Predicted Bay Frequency (f_n^4)**



**(c) Measured Bay Frequency vs. SCI
Predicted Bay Frequency (f_n^6)**

**Figure 2.6 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors
with Hot-Rolled Shapes**

2.3.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS

Table 2.6 summarizes the results from the frequency comparisons for floors that are constructed of hot-rolled girders and joist beams. According to SCI predictions, the ratio comparing predicted beam and measured bay frequencies demonstrates that the secondary framing does not solely control the floor behavior. From Table 2.6, the average ratio between predicted beam and measured bay frequencies (f_b/f_m) is 1.41 with a high standard deviation of 0.73. The average ratio between predicted girder and measured bay frequencies (f_g/f_m) is 0.95 with a modest standard deviation of 0.15. The average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.99 with a standard deviation of 0.22. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , marginally provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 1.01 with a standard deviation of 0.21. The average f_n^6/f_m ratio is 1.01 with a standard deviation of 0.21.

The measured bay frequencies are plotted versus predicted bay frequencies, f_n^2 , in Figure 2.7(a). This figure illustrates that the predicted bay frequencies are within 33 % of the measured bay frequencies for only about 38 % of the case studies with hot-rolled girders and joist beams. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.7(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 33 % of the case studies with hot-rolled girders and joist beams. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.7(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 33 % of the case studies with hot-rolled girders and joist beams.

Table 2.6 – SCI Frequency Comparison for Floors with Hot-Rolled Girders and Joists

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Premiere Club	351	1	23.35	6.92	6.63	6.91	6.92	6.00	3.892	1.153	1.105	1.151	1.153
Patterson Avenue Facility	549	4	6.58	N/A	6.58	6.58	6.58	4.50	1.462	N/A	1.462	1.462	1.462
		5	6.58	N/A	6.58	6.58	6.58	4.50	1.462	N/A	1.462	1.462	1.462
		7	5.47	N/A	5.47	5.47	5.47	4.50	1.216	N/A	1.216	1.216	1.216
Orange Synagogue	349	1	16.03	6.91	6.35	6.85	6.90	7.50	2.137	0.921	0.847	0.914	0.920
		4	16.03	6.91	6.35	6.85	6.90	8.50	1.886	0.813	0.747	0.806	0.812
		5	16.03	6.91	6.35	6.85	6.90	8.00	2.004	0.864	0.794	0.856	0.863
ACTIC Building	341	Typ.	6.21	N/A	6.21	6.21	6.21	6.50	0.955	N/A	0.955	0.955	0.955
Camron University	514	271	6.16	N/A	6.16	6.16	6.16	7.00	0.880	N/A	0.880	0.880	0.880
		273	5.88	N/A	5.88	5.88	5.88	6.75	0.871	N/A	0.871	0.871	0.871
Melroe Corp.	508	Typ.	12.21	4.93	4.57	4.90	4.93	5.50	2.220	0.896	0.831	0.891	0.896
Steel Case	570	Mezz.	11.73	4.73	4.39	4.70	4.73	4.75	2.469	0.996	0.924	0.989	0.995
		Int.	11.73	3.62	3.46	3.61	3.62	4.75	2.469	0.762	0.728	0.760	0.762
UT Medical Center	Boice1	A	4.53	N/A	4.53	4.53	4.53	5.00	0.906	N/A	0.906	0.906	0.906
		B	4.53	N/A	4.53	4.53	4.53	4.75	0.954	N/A	0.954	0.954	0.954
InterqQuest/ Ford	615	Typ.	11.87	5.56	5.04	5.50	5.55	4.75	2.499	1.171	1.061	1.157	1.168
XILINX Building	609	Typ.	5.05	N/A	5.05	5.05	5.05	4.75	1.063	N/A	1.063	1.063	1.063
CO Civic Center	610	1	5.16	N/A	5.16	5.16	5.16	5.25	0.983	N/A	0.983	0.983	0.983
		2	5.26	N/A	5.26	5.26	5.26	5.25	1.002	N/A	1.002	1.002	1.002
		3	4.70	N/A	4.70	4.70	4.70	5.50	0.855	N/A	0.855	0.855	0.855
Gateway 800 Bridge	Boice7	Typ.	2.52	N/A	2.52	2.52	2.52	4.00	0.630	N/A	0.630	0.630	0.630
Ruby Tuesday	Slad1	2nd.	4.49	N/A	4.49	4.49	4.49	4.25	1.056	N/A	1.056	1.056	1.056
		3rd.	3.86	N/A	3.86	3.86	3.86	3.75	1.029	N/A	1.029	1.029	1.029
MidWest Kentworth	Band3	Typ.	4.04	N/A	4.04	4.04	4.04	5.75	0.703	N/A	0.703	0.703	0.703
Nations Bank	Band4	1,4	4.04	N/A	4.04	4.04	4.04	3.85	1.049	N/A	1.049	1.049	1.049
		2,6	3.53	N/A	3.53	3.53	3.53	5.00	0.706	N/A	0.706	0.706	0.706
		3	3.53	N/A	3.53	3.53	3.53	3.90	0.905	N/A	0.905	0.905	0.905
		5	4.04	N/A	4.04	4.04	4.04	3.40	1.188	N/A	1.188	1.188	1.188
Eight Parkwood	618	2	7.07	N/A	7.07	7.07	7.07	5.25	1.347	N/A	1.347	1.347	1.347
		6	7.07	N/A	7.07	7.07	7.07	5.50	1.285	N/A	1.285	1.285	1.285

Average -	1.412	0.947	0.994	1.011	1.012
Standard Deviation -	0.731	0.150	0.217	0.211	0.210

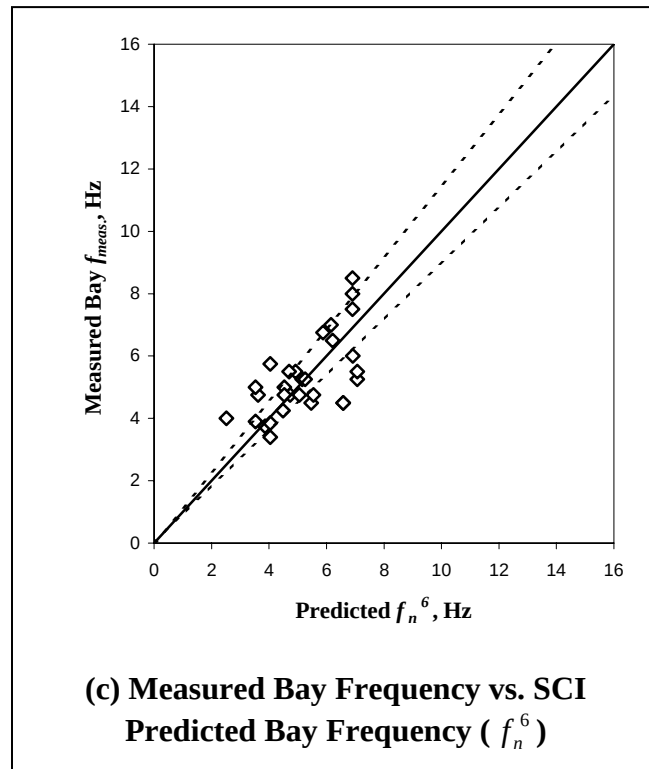
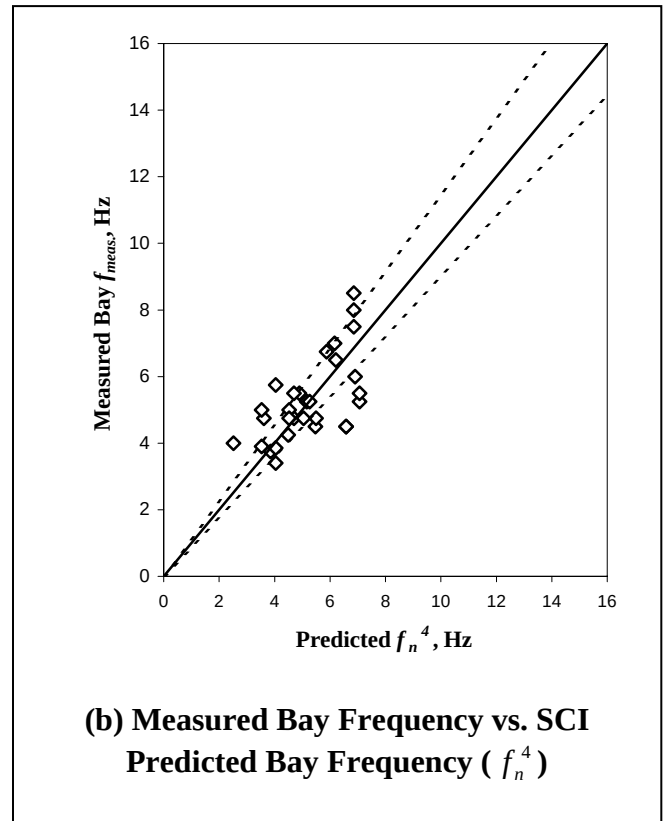
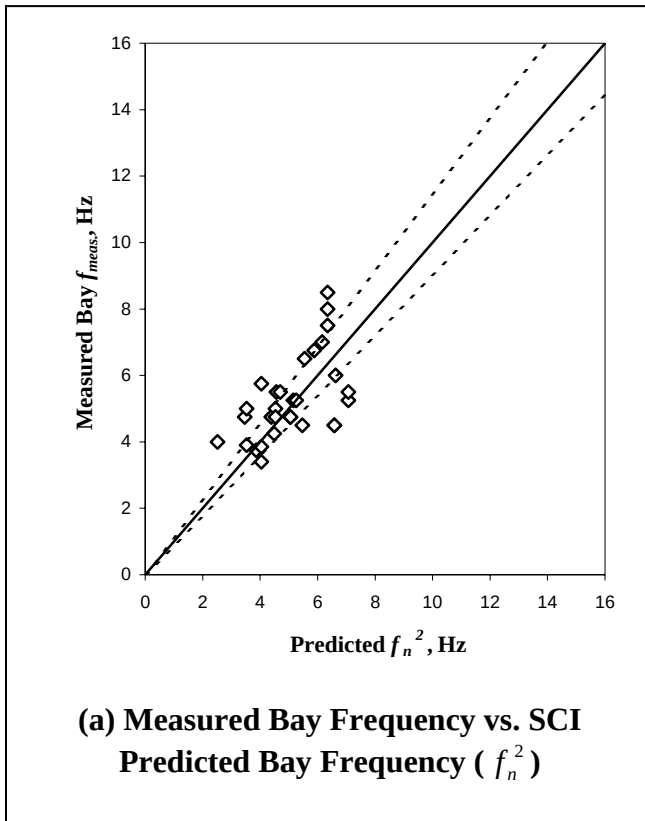


Figure 2.7 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Hot-Rolled Girders and Joists

2.3.3 FLOORS WITH JOIST GIRDERS AND JOISTS

Table 2.7 summarizes the results from the frequency comparisons for floors that are constructed of joist girders and joist beams. From Table 2.7, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 2.03 with a high standard deviation of 0.79. The average ratio between predicted girder and measured frequencies (f_g/f_m) is 0.83 with a modest standard deviation of 0.18. The average ratio of predicted system frequency to measured frequency (f_n^2/f_m) is 0.78 with a standard deviation of 0.17. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , marginally provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 0.82 with a standard deviation of 0.18. The average f_n^6/f_m ratio is 0.82 with a standard deviation of 0.18.

Table 2.7 – SCI Frequency Comparison for Floors with Joist Girders and Joists

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Z.C.M.I. Store	269	1-4	16.18	5.45	5.16	5.43	5.45	5.65	2.864	0.965	0.913	0.962	0.964
		5,10	14.99	4.57	4.37	4.56	4.57	4.75	3.156	0.962	0.920	0.960	0.962
		9	17.05	5.82	5.51	5.80	5.82	7.04	2.422	0.827	0.783	0.824	0.826
		7,8,11	16.18	5.45	5.16	5.43	5.45	5.11	3.166	1.067	1.010	1.063	1.066
Patterson	549	6	13.50	5.03	4.72	5.01	5.03	5.00	2.700	1.006	0.944	1.001	1.006
Toyota	481	Typ.	4.94	N/A	4.94	4.94	4.94	5.63	0.877	N/A	0.877	0.877	0.877
Garmin Int. Bldg.	506	Int.	13.97	4.10	3.94	4.09	4.10	6.25	2.235	0.656	0.630	0.655	0.656
		Ext.	13.97	4.10	3.94	4.09	4.10	6.25	2.235	0.656	0.630	0.655	0.656
Sony Corp.	511	Typ.	12.24	4.04	3.83	4.03	4.04	5.50	2.225	0.735	0.696	0.732	0.734
Diversa Building	568	Ext.-1	13.69	5.61	5.19	5.57	5.61	5.75	2.381	0.976	0.903	0.969	0.975
		Ext.-2	10.54	4.35	4.02	4.32	4.35	5.00	2.108	0.870	0.804	0.864	0.869
		Int.-1	21.16	6.16	5.91	6.15	6.16	9.50	2.227	0.648	0.622	0.647	0.648
		Int.-2	16.09	4.71	4.52	4.70	4.71	13.00	1.238	0.362	0.348	0.362	0.362
Pacific	568	Typ.	13.73	5.83	5.37	5.78	5.82	6.00	2.288	0.972	0.895	0.964	0.971
IBM	605	Typ.	6.78	N/A	6.78	6.78	6.78	6.00	1.130	N/A	1.130	1.130	1.130
Southern Bell	Band1	2	12.27	5.69	5.16	5.63	5.68	6.25	1.963	0.910	0.826	0.900	0.909
		5,7	12.27	5.69	5.16	5.63	5.68	6.75	1.818	0.843	0.764	0.833	0.842
		9	16.78	5.79	5.48	5.77	5.79	9.50	1.766	0.609	0.577	0.607	0.609
Wholesale Grocers	Band2	1	2.45	N/A	2.45	2.45	2.45	3.50	0.700	N/A	0.700	0.700	0.700
		3	2.53	N/A	2.53	2.53	2.53	4.25	0.595	N/A	0.595	0.595	0.595
		5	2.03	N/A	2.03	2.03	2.03	3.00	0.677	N/A	0.677	0.677	0.677
Eight Parkwood	618	1	13.66	4.52	4.29	4.51	4.52	5.25	2.602	0.861	0.817	0.858	0.861
		3	13.66	4.73	4.47	4.71	4.73	5.75	2.376	0.823	0.777	0.820	0.822
		4,5	13.66	4.52	4.29	4.51	4.52	4.50	3.036	1.004	0.953	1.001	1.004
R.S.I. Building	621	1	8.54	N/A	8.54	8.54	8.54	9.50	0.899	N/A	0.899	0.899	0.899
		2	8.54	N/A	8.54	8.54	8.54	9.75	0.876	N/A	0.876	0.876	0.876
		3	9.61	N/A	9.61	9.61	9.61	9.75	0.986	N/A	0.986	0.986	0.986
		4	6.87	N/A	6.87	6.87	6.87	7.25	0.948	N/A	0.948	0.948	0.948

Average -	2.033	0.829	0.783	0.819	0.822
Standard Deviation -	0.786	0.178	0.170	0.181	0.182

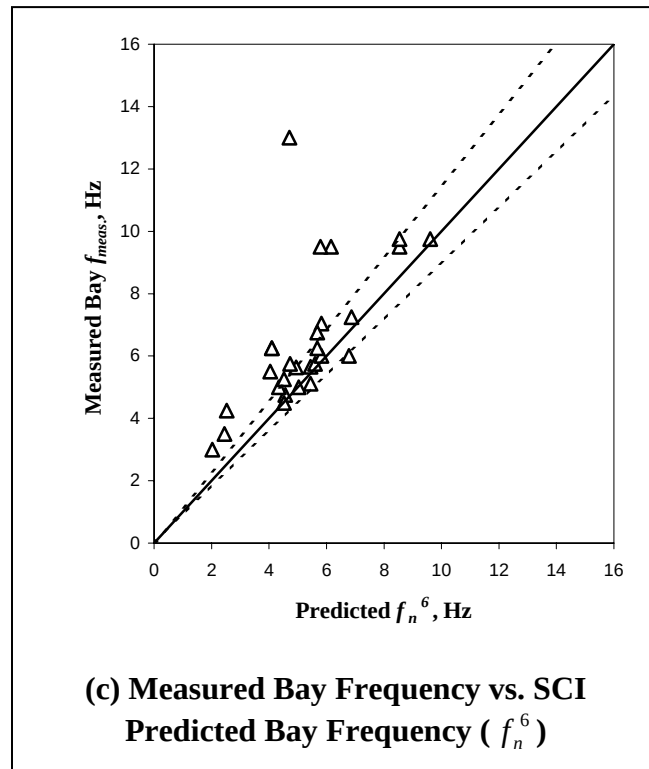
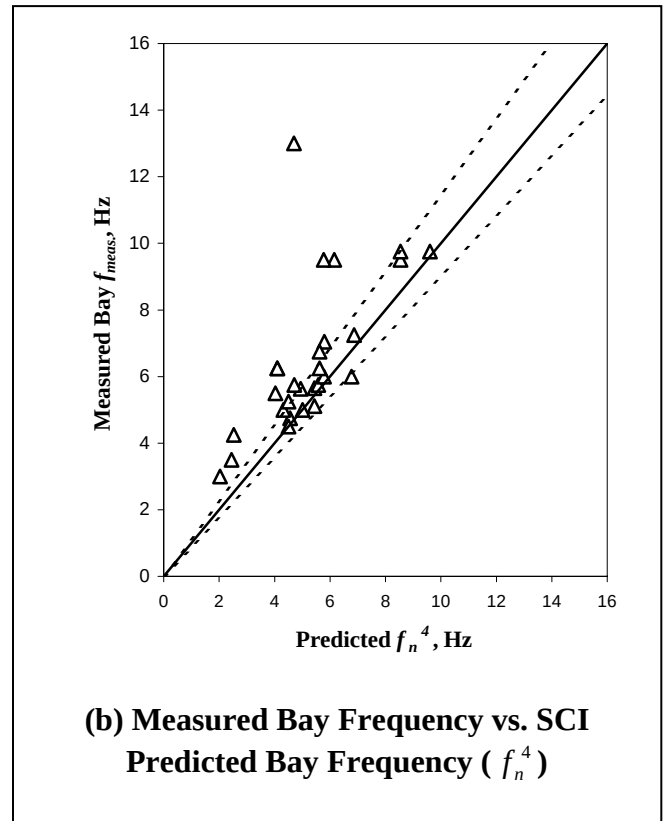
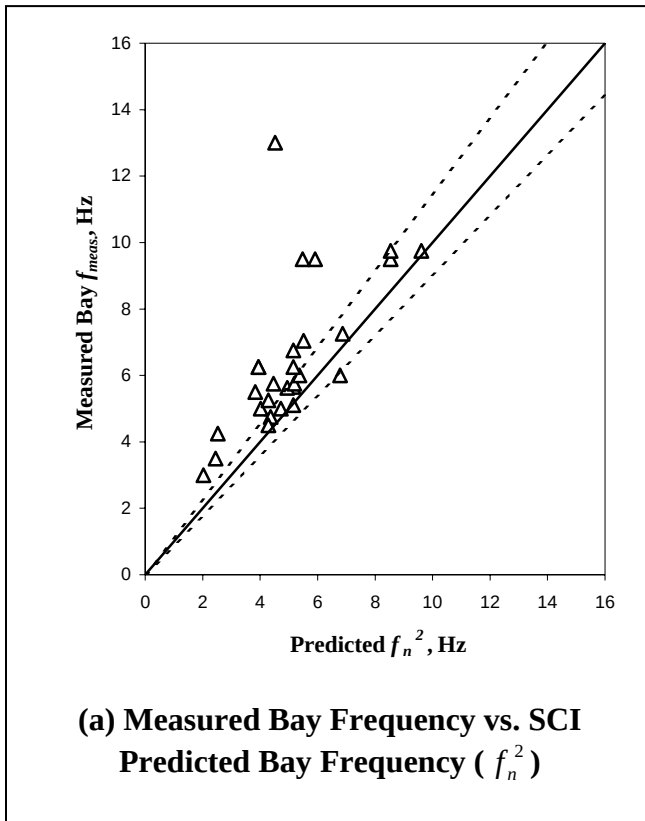


Figure 2.8 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Joist Girders and Joists

The measured bay frequencies are plotted versus predicted bay frequencies, f_n^2 , in Figure 2.8(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 29 % of the case studies with joist girders and joist beams. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.8(b). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 36 % of the case studies with joist girders and joist framing. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.8(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for about 36 % of the case studies with joist girders and joist beams.

2.3.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS

Table 2.8 summarizes the results from the frequency comparison for floors that are constructed of hot-rolled and castellated framing. From Table 2.8, the average ratio of predicted bay frequency to measured frequency (f_n^2/f_m) is 0.99 with a low standard deviation of 0.03. The average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.82 with a high standard deviation of 0.74. The comparison between measured and predicted bay frequencies, f_n^4 and f_n^6 , provides more reliable correlations than estimations using f_n^2 . The average f_n^4/f_m ratio is 1.04 with a standard deviation of 0.05. The average f_n^6/f_m ratio is 1.05 with a standard deviation of 0.05. The average f_g/f_m ratio is 1.10 with a standard deviation of 0.04.

Table 2.8 – SCI Frequency Comparison for Floors with Hot-Rolled Girders and Castellated Beams

Building	File	Bay	Predicted Frequencies					Meas. f_m (Hz)	Frequency Ratios				
			f_b (Hz)	f_g (Hz)	f_n^2 (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_b/f_m	f_g/f_m	f_n^2/f_m	f_n^4/f_m	f_n^6/f_m
Lennox	572	1,3	3.41	N/A	3.41	3.41	3.41	3.32	1.029	N/A	1.029	1.029	1.029
Wal-Mart	572	4,5	4.66	N/A	4.66	4.66	4.66	4.75	0.981	N/A	0.981	0.981	0.981
AMAT	Boice7	B,B2	15.50	7.56	6.80	7.46	7.54	6.75	2.296	1.120	1.007	1.105	1.118
		C	11.35	5.02	4.59	4.97	5.01	4.75	2.389	1.057	0.966	1.047	1.056
		D	15.50	6.95	6.34	6.88	6.94	6.50	2.385	1.069	0.975	1.059	1.068

Average -	1.816	1.082	0.992	1.044	1.05
Standard Deviation -	0.742	0.033	0.026	0.045	0.050

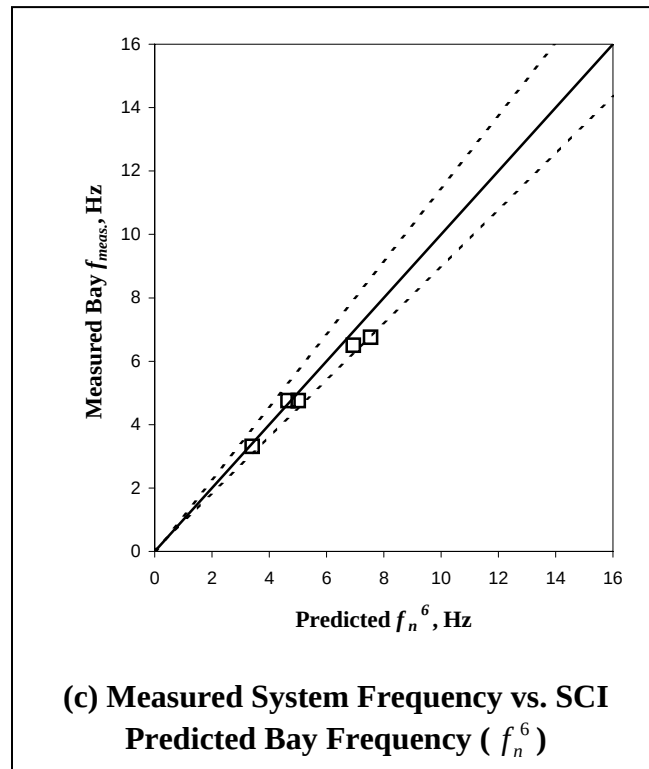
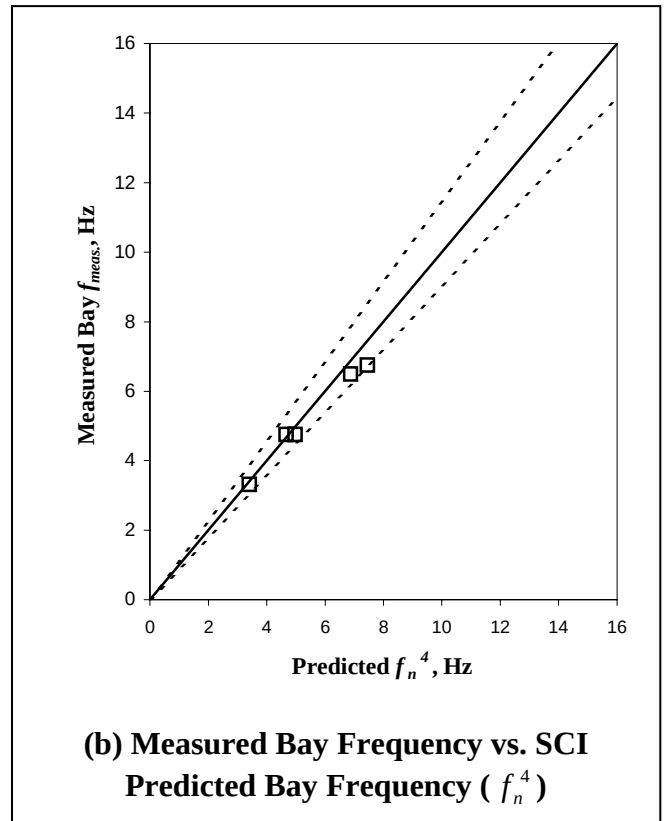
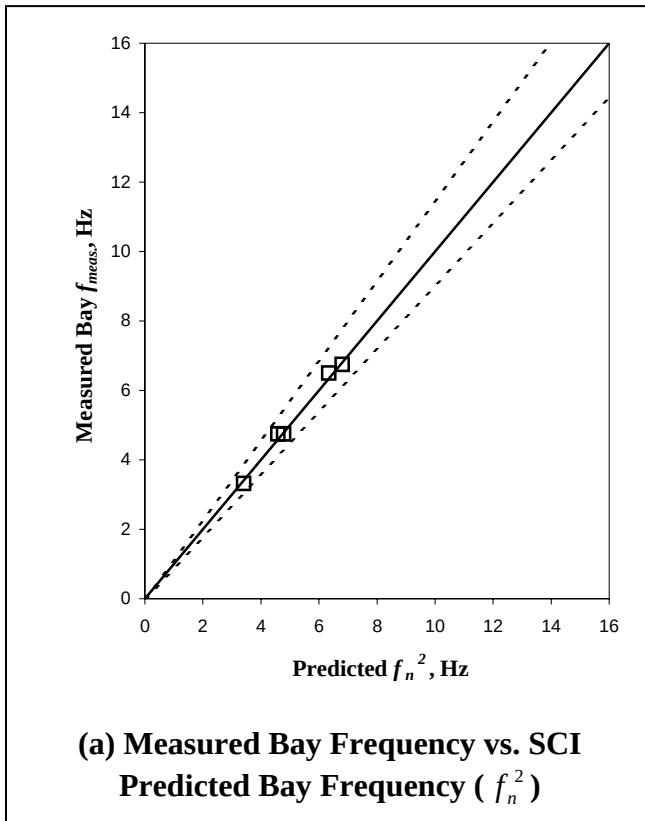


Figure 2.9 – Measured Bay Frequencies vs. SCI Predicted Frequencies for Floors with Hot-Rolled Girders and Castelled Beams

The measured bay frequencies are plotted versus predicted bay frequencies, f_n^2 , in Figure 2.9(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for 100 % of the case studies with hot-rolled girders and castellated beams. The measured bay frequencies are plotted against the predicted bay frequencies, using the revision f_n^4 , in Figure 2.9(b). This figure illustrates that the predicted system frequencies are within 10 % of the measured bay frequencies for about 80 % of the case studies with hot-rolled girders and castellated beams. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.9(c). This figure illustrates that the predicted bay frequencies are within 10 % of the measured system frequencies for about 80 % of the case studies with hot-rolled girders and castellated beams.

2.3.5 SUMMARY OF SCI FREQUENCY COMPARISONS

Based on a statistical analysis of the data in Tables 2.5 through 2.8, the average ratio between predicted beam and measured frequencies (f_b/f_m) is 1.64 with a high standard deviation of 0.77 for floors constructed of all types of framing, meaning that the secondary framing does not solely control floor behavior when the SCI method is used. The f_g/f_m ratio is 0.90 with a modest standard deviation of 0.18. The average ratio of predicted system frequency to measured frequency (f_n^2/f_m) is 0.93 with a modest standard deviation of 0.20. The average f_n^4/f_m ratio is 0.95 with a standard deviation of 0.20. Collectively, the average f_n^6/f_m ratio is 0.95 with a standard deviation of 0.20 for floors constructed of all types of framing.

The measured bay frequencies are plotted against the predicted bay frequencies, f_n^2 , in Figure 2.10(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only about 38 % of case studies. From Figure 2.10(b), the predicted bay frequencies, using the revision f_n^4 , are within 10 % of the measured bay frequencies for about 43 % of the case studies. The measured frequencies are plotted against the predicted bay frequencies, using the revision f_n^6 , in Figure 2.10(c). This figure also illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for 43 % of the case studies.

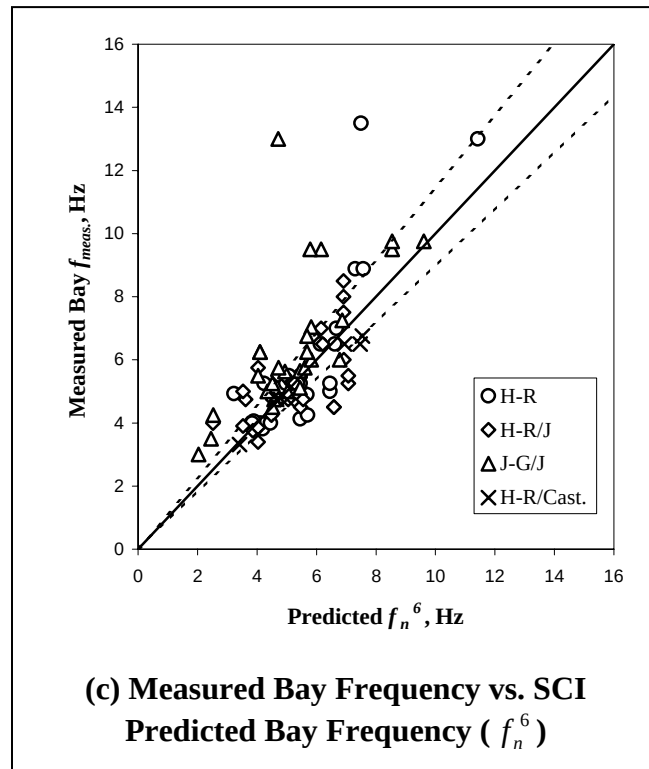
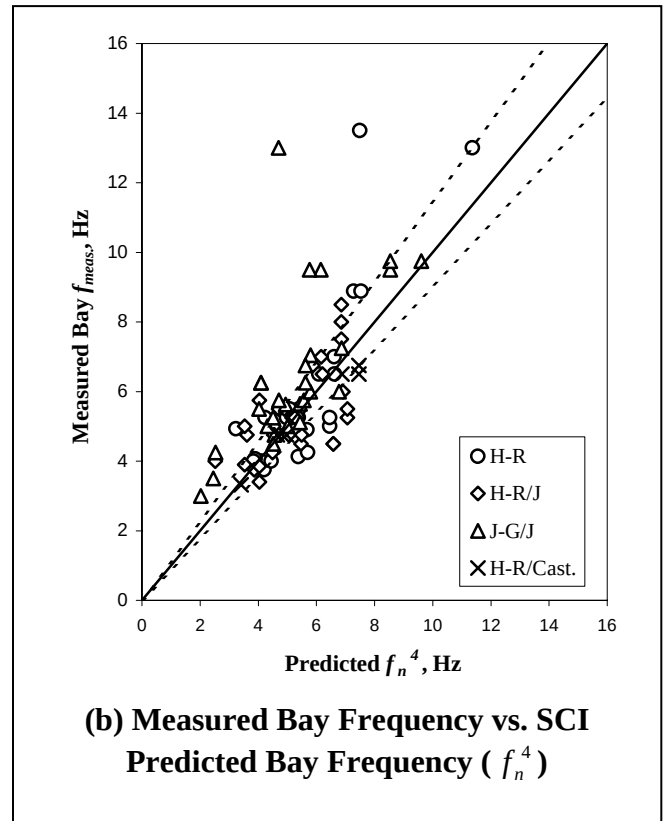
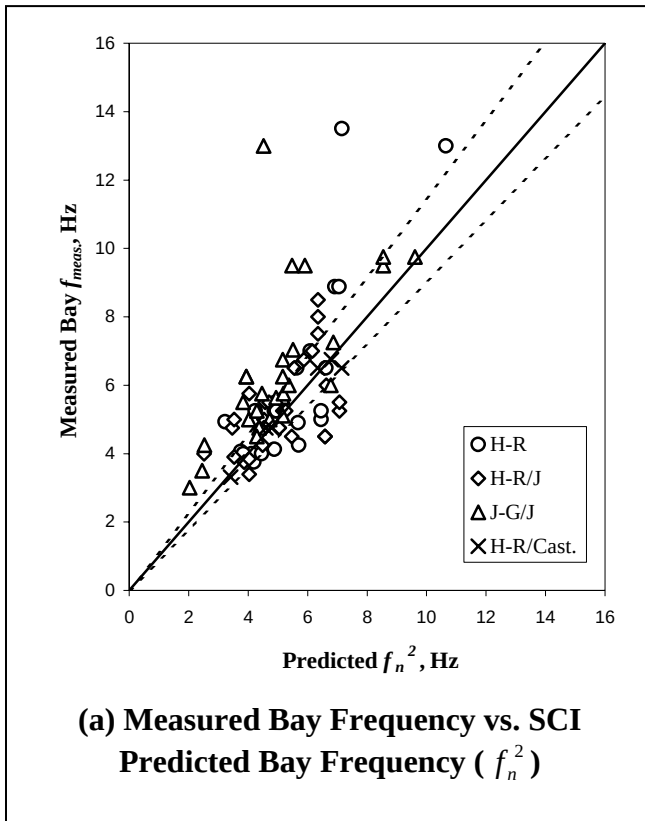


Figure 2.10 – Measured Bay Frequencies vs. SCI Predicted Frequencies for All Framing Types

2.4 AISC vs. SCI FREQUENCY COMPARISON

This section focuses on comparing the methods provided by AISC and SCI for predicting the fundamental natural frequency of structural floor systems. The theoretical or predicted bay frequencies are compared with measured system frequencies for the 89 case studies evaluated in Sections 2.2 and 2.3, which include all types of framing. Tables 2.9 through 2.12 summarize the comparison between measured and predicted bay frequencies using AISC and SCI techniques for the four framing types. The predicted system frequency of interest for comparison is Dunkerly's estimate, f_n^2 . In Tables 2.9 through 2.12, f_n^2 is replaced with f_{bay} . Predicted beam and girder frequencies are not evaluated in this section due to poor correlation with measured bay frequencies as shown in the previous sections.

2.4.1 FLOORS WITH HOT-ROLLED SHAPES

Table 2.9 summarizes the results from the frequency comparison between AISC and SCI for floors that are constructed of hot-rolled framing. From Table 2.9, the average ratio of predicted bay frequency using the AISC Design Guide to measured frequency (f_{bay}/f_m) is 0.80 with a modest standard deviation of 0.13. The average ratio of predicted bay frequency using the SCI method to measured frequency (f_{bay}/f_m) is 0.96 with a modest standard deviation of 0.19.

2.4.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS

Table 2.10 summarizes the results from the frequency comparison between AISC and SCI for floors that are constructed of hot-rolled girders and joist beams. From Table 2.10, the average ratio of predicted bay frequency using the AISC Design Guide to measured frequency (f_{bay}/f_m) is 0.90 with a standard deviation of 0.20. The average ratio of predicted bay frequency using the SCI method to measured frequency (f_{bay}/f_m) is 0.99 with a standard deviation of 0.22.

2.4.3 FLOORS WITH JOIST GIRDERS AND JOISTS

Table 2.11 summarizes the results from the frequency comparison between AISC and SCI for floors that are constructed of joist girder and joist framing. From Table 2.11, the average ratio of predicted bay frequency using the AISC Design Guide to measured

frequency (f_{bay}/f_m) is 0.76 with a modest standard deviation of 0.17. The average ratio of predicted bay frequency using the SCI method to measured frequency (f_{bay}/f_m) is 0.78 with a modest standard deviation of 0.17.

Table 2.9 – AISC vs. SCI Frequency Comparison for Floors with Hot-Rolled Shapes

Building	File	Bay	Predicted Frequencies						Meas. f_m (Hz)	Frequency Ratios		
			AISC			SCI				AISC	SCI	AISC/SCI
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_b (Hz)	f_g (Hz)	f_{bay} (Hz)		f_{bay}/f_m	f_{bay}/f_m	$f_{bay}^{AISC} / f_{bay}^{SCI}$
Jersey Lils	272	Typ.	3.23	10.40	3.15	3.23	N/A	3.23	4.93	0.639	0.655	0.975
Mercy Health Ctr.	508	Int. Ext.	9.97	7.19	6.09	21.58	7.31	6.92	8.88	0.686	0.779	0.880
			8.92	7.58	6.09	19.44	7.58	7.06	8.88	0.686	0.795	0.863
Iowa State	535	1	4.07	5.15	3.19	4.07	N/A	4.07	3.88	0.822	1.049	0.784
		2,3	4.12	9.11	3.82	4.20	N/A	4.20	3.75	1.019	1.120	0.910
		4	4.12	9.13	3.82	4.12	N/A	4.12	4.00	0.955	1.030	0.927
Oracle Building	565	1,2	4.87	5.83	3.81	10.89	5.47	4.89	5.25	0.726	0.931	0.779
		3,4	4.87	5.83	3.81	10.89	5.47	4.89	4.13	0.923	1.184	0.779
		5	5.15	5.69	3.88	11.46	5.47	4.93	5.50	0.705	0.896	0.787
Newgen	568	Ext.	6.62	9.53	5.47	6.62	N/A	6.62	6.50	0.842	1.018	0.826
42nd St.	575	Typ.	6.50	3.95	3.38	14.37	3.90	3.76	4.06	0.833	0.926	0.899
Penn State	Boice3	Typ.	4.51	8.66	4.23	5.68	N/A	5.68	4.90	0.863	1.159	0.745
Newport V	600	Typ.	3.78	6.80	3.38	3.84	N/A	3.84	4.00	0.845	0.960	0.880
Doylestown	569	Typ.	6.86	6.98	5.19	14.94	6.67	6.09	7.00	0.741	0.870	0.852
UT Med. Center	Boice1	C	13.47	11.96	8.94	29.33	11.43	10.65	13.00	0.688	0.819	0.839
		D	12.73	7.53	6.48	28.40	7.50	7.16	13.50	0.480	0.530	0.905
One Texas Court	617	1	4.83	6.29	3.96	4.97	N/A	4.97	5.25	0.754	0.947	0.797
		2	6.54	6.36	4.61	14.33	6.14	5.64	6.50	0.709	0.868	0.817
AMEX	602	Typ.	4.24	0.00	4.24	4.24	N/A	4.24	5.25	0.808	0.808	1.000
GTE	Slad2	Typ.	5.06	6.17	4.00	5.08	N/A	5.08	5.50	0.727	0.924	0.787
Three Parkwood	618	1	5.35	6.79	4.20	5.70	N/A	5.70	4.25	0.988	1.341	0.737
		2	4.62	5.77	3.65	4.45	N/A	4.45	4.00	0.913	1.113	0.820
Norman Pointe 1	620	1	6.05	7.97	4.91	6.46	N/A	6.46	5.00	0.982	1.292	0.760
		3	5.12	4.53	3.39	12.57	5.38	4.95	5.25	0.646	0.943	0.685
		4	6.05	7.97	4.91	6.46	N/A	6.46	5.25	0.935	1.230	0.760

Average -	0.797	0.963	0.835
Standard Deviation -	0.132	0.194	0.072

2.4.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS

Table 2.12 summarizes the results from the frequency comparison between AISC and SCI for floors that are constructed of hot-rolled girders and castellated framing. From Table 2.12, the average ratio of predicted bay frequency using the AISC Design Guide to measured frequency (f_{bay}/f_m) is 0.86 with a standard deviation of 0.09. The average ratio of predicted bay frequency using the SCI method to measured frequency (f_{bay}/f_m) is 0.99 with a small standard deviation of 0.03.

Table 2.10 – AISC vs. SCI Frequencies for Hot-Rolled Girders and Joists

Building	File	Bay	Predicted Frequencies						Meas. f_m (Hz)	Frequency Ratios		
			AISC			SCI				AISC	SCI	AISC/SCI
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_b (Hz)	f_g (Hz)	f_{bay} (Hz)		f_{bay}/f_m	f_{bay}/f_m	$f_{bay}^{AISC} / f_{bay}^{SCI}$
Premiere Club	351	1	10.67	7.08	5.90	23.35	6.92	6.63	6.00	0.983	1.105	0.890
Patterson Avenue Facility	549	4	6.58	12.57	5.97	6.58	N/A	6.58	4.50	1.327	1.462	0.907
		5	6.58	10.41	5.75	6.58	N/A	6.58	4.50	1.278	1.462	0.874
		7	5.51	7.51	4.68	5.47	N/A	5.47	4.50	1.040	1.216	0.856
Orange Synagogue	349	1	7.10	7.57	5.36	16.03	6.91	6.35	7.50	0.715	0.847	0.844
		4	7.10	7.57	5.36	16.03	6.91	6.35	8.50	0.631	0.747	0.844
		5	7.10	7.57	5.36	16.03	6.91	6.35	8.00	0.670	0.794	0.844
ACTIC	341	Typ.	6.21	12.40	5.64	6.21	N/A	6.21	6.50	0.868	0.955	0.908
Camron University	514	271	6.16	10.98	5.65	6.16	N/A	6.16	7.00	0.807	0.880	0.917
		273	5.88	10.50	5.39	5.88	N/A	5.88	6.75	0.799	0.871	0.917
Melroe Corp.	508	Typ.	5.46	5.28	3.80	12.21	4.93	4.57	5.50	0.691	0.831	0.832
Steel Case	570	Mezz. Int.	5.25	4.92	3.59	11.73	4.73	4.39	4.75	0.756	0.924	0.818
			5.25	4.02	3.19	11.73	3.62	3.46	4.75	0.672	0.728	0.922
UT Medical	Boice1	A	5.23	9.29	4.76	4.53	N/A	4.53	5.00	0.952	0.906	1.051
		B	5.22	9.16	4.73	4.53	N/A	4.53	4.75	0.996	0.954	1.044
IQ/ Ford	615	Typ.	5.31	5.96	3.97	11.87	5.56	5.04	4.75	0.836	1.061	0.788
XILINX	609	Typ.	5.05	6.06	3.93	5.05	N/A	5.05	4.75	0.827	1.063	0.778
CO Civic Center	610	1	5.16	N/A	5.16	5.16	N/A	5.16	5.25	0.983	0.983	1.000
		2	5.26	N/A	5.26	5.26	N/A	5.26	5.25	1.002	1.002	1.000
		3	4.70	N/A	4.70	4.70	N/A	4.70	5.50	0.855	0.855	1.000
Gateway 800 Bridge	Boice2	Typ.	2.52	4.07	2.22	2.52	N/A	2.52	4.00	0.555	0.630	0.881
Ruby Tuesday	Slad1	2nd.	4.49	10.50	4.26	4.49	N/A	4.49	4.25	1.002	1.056	0.949
		3rd.	3.86	10.56	3.72	3.86	N/A	3.86	3.75	0.992	1.029	0.964
Midwest Kentworth	Band3	Typ.	4.04	0.00	4.04	4.04	N/A	4.04	5.75	0.703	0.703	1.000
Nations Bank	Band4	1,4	4.04	8.09	3.77	4.04	N/A	4.04	3.85	0.979	1.049	0.933
		2,6	3.53	8.09	3.36	3.53	N/A	3.53	5.00	0.672	0.706	0.952
		3	3.53	8.09	3.36	3.53	N/A	3.53	3.90	0.862	0.905	0.952
		5	4.04	8.09	3.77	4.04	N/A	4.04	3.40	1.109	1.188	0.933
Eight Parkwood	618	2	7.61	10.82	6.22	7.07	N/A	7.07	5.25	1.185	1.347	0.880
		6	7.61	10.82	6.22	7.07	N/A	7.07	5.50	1.131	1.285	0.880

Average -	0.896	0.994	0.903
Standard Deviation -	0.195	0.217	0.061

Table 2.11 – AISC vs. SCI Frequencies for Floors with Joist Girders and Joists

Building	File	Bay	Predicted Frequencies						Meas. f_m (Hz)	Frequency Ratios		
			AISC			SCI				AISC	SCI	AISC/SCI
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_b (Hz)	f_g (Hz)	f_{bay} (Hz)		f_{bay}/f_m	f_{bay}/f_m	$f_{bay}^{AISC} / f_{bay}^{SCI}$
Z.C.M.I. Store	269	1--4	7.23	6.22	4.72	16.18	5.45	5.16	5.65	0.835	0.913	0.915
		5,10	6.86	4.99	4.03	14.99	4.57	4.37	4.75	0.848	0.920	0.922
		9	7.62	6.63	5.00	17.05	5.82	5.51	7.04	0.710	0.783	0.907
		7,8,11	7.23	6.22	4.72	16.18	5.45	5.16	5.11	0.924	1.010	0.915
Patterson	549	6	6.04	6.10	4.67	13.50	5.03	4.72	5.00	0.934	0.944	0.989
Toyota	481	Typ.	4.94	8.02	4.53	4.94	N/A	4.94	5.63	0.805	0.877	0.917
Garmin Int. Bldg.	506	Int.	6.25	4.52	3.66	13.97	4.10	3.94	6.25	0.586	0.630	0.929
		Ext.	6.25	4.52	3.66	13.97	4.10	3.94	6.25	0.586	0.630	0.929
Sony Corp.	511	Typ.	5.47	4.44	3.45	12.24	4.04	3.83	5.50	0.627	0.696	0.901
Diversa Building	568	Ext.-1	6.12	6.45	4.44	13.69	5.61	5.19	5.75	0.772	0.903	0.855
		Ext.-2	4.71	5.00	3.43	10.54	4.35	4.02	5.00	0.686	0.804	0.853
		Int.-1	9.52	7.29	5.79	21.16	6.16	5.91	9.50	0.609	0.622	0.980
		Int.-2	7.24	5.57	4.42	16.09	4.71	4.52	13.00	0.340	0.348	0.978
Pacific	568	Typ.	6.14	6.71	4.53	13.73	5.83	5.37	6.00	0.755	0.895	0.844
IBM	605	Typ.	6.78	N/A	6.78	6.78	N/A	6.78	6.00	1.130	1.130	1.000
Southern Bell	Band1	2	5.54	6.67	4.48	12.27	5.69	5.16	6.25	0.717	0.826	0.868
		5,7	5.54	6.67	4.48	12.27	5.69	5.16	6.75	0.664	0.764	0.868
		9	7.58	6.79	5.22	16.78	5.79	5.48	9.50	0.549	0.577	0.953
Wholesale Grocers	Band2	1	2.45	N/A	2.45	2.45	N/A	2.45	3.50	0.700	0.700	1.000
		3	2.53	N/A	2.53	2.53	N/A	2.53	4.25	0.595	0.595	1.000
		5	2.03	N/A	2.03	2.03	N/A	2.03	3.00	0.677	0.677	1.000
Eight Parkwood	618	1	6.11	5.89	4.24	13.66	4.52	4.29	5.25	0.808	0.817	0.988
		3	6.11	6.10	4.32	13.66	4.73	4.47	5.75	0.751	0.777	0.966
		4,5	6.11	5.89	4.24	13.66	4.52	4.29	4.50	0.942	0.953	0.988
R.S.I. Building	621	1	8.54	N/A	8.54	8.54	N/A	8.54	9.50	0.899	0.899	1.000
		2	8.54	N/A	8.54	8.54	N/A	8.54	9.75	0.876	0.876	1.000
		3	9.61	N/A	9.61	9.61	N/A	9.61	9.75	0.986	0.986	1.000
		4	6.87	N/A	6.87	6.87	N/A	6.87	7.25	0.948	0.948	1.000

Average -	0.759	0.783	0.936
Standard Deviation -	0.166	0.170	0.053

Table 2.12 – AISC vs. SCI Frequencies for Hot-Rolled Girders with Castellated Beams

Building	File	Bay	Predicted Frequencies						Meas.	Frequency Ratios		
			AISC			SCI				AISC	SCI	AISC/SCI
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_b (Hz)	f_g (Hz)	f_{bay} (Hz)		f_{bay}/f_m	f_{bay}/f_m	$f_{bay}^{AISC} / f_{bay}^{SCI}$
Lennox	572	1,3	3.41	7.16	3.19	3.41	N/A	3.41	3.32	0.962	1.029	0.935
Wal-Mart	572	4,5	4.66	10.22	4.44	4.66	N/A	4.66	4.75	0.935	0.981	0.953
AMAT	Boice7	B,B2	6.93	7.88	5.21	15.50	7.56	6.80	6.75	0.772	1.007	0.766
		C	5.63	5.79	4.04	11.35	5.02	4.59	4.75	0.851	0.966	0.880
		D	6.93	7.21	5.00	15.50	6.95	6.34	6.50	0.769	0.975	0.789

Average -	0.858	0.992	0.865
Standard Deviation -	0.090	0.026	0.084

2.5 FREQUENCY EVALUATION OF BRITISH FLOORS

This section focuses on comparing the methods provided by AISC and SCI for predicting the fundamental natural frequency of structural floor systems. Fourteen case studies of floors previously evaluated in the SCI publication *Design Guide for Vibration of Long Span Composite Floors* (Hicks, et al. 2000) are assessed. These floors use hot-rolled, castellated, or cellular framing. Cellular beams are similar to castellated beams in that the cross sectional properties for both are non-prismatic. Therefore, net rather than gross section properties were used when modeling the floor systems.

Again, predicted system frequencies based on the AISC Design Guide are calculated using the techniques discussed in Section 2.2. SCI predictions of system frequencies are calculated using the techniques presented in Section 1.4.3. Tables 2.13 and 2.14 summarize the comparison between measured and predicted bay frequencies using the AISC and SCI methods. Measured frequencies are plotted against predicted system frequencies in Figure 2.11. Again, the predicted frequencies that are of interest for comparison are Dunkerly's estimate, f_n^2 , as well as approximations using f_n^4 and f_n^6 . Predicted beam and girder frequencies are not evaluated in this section due to poor correlation with measured bay frequencies. In Tables 2.13 and 2.14, f_n^2 is replaced with f_{bay} . In addition, system properties are investigated and compared for an example floor system originally tested by SCI. Specifically, differences in effective width and composite moment of inertia values are examined.

2.5.1 AISC FREQUENCY EVALUATION OF BRITISH FLOORS

The AISC Design Guide proved to be a fairly accurate tool for predicting the fundamental natural frequencies of the British floor systems. From Table 2.13, the average ratio of predicted system frequency to measured frequency (f_{bay}/f_m) is 0.93. However, a modest standard deviation of 0.18 suggests that the data is moderately scattered. The average ratio of predicted system frequency to measured frequency (f_n^4/f_m) is 1.00 with a standard deviation of 0.15, and for (f_n^6/f_m) is 1.02 with a modest standard deviation of 0.15.

Table 2.13 – System Frequency Comparison for British Floors Using AISC Method

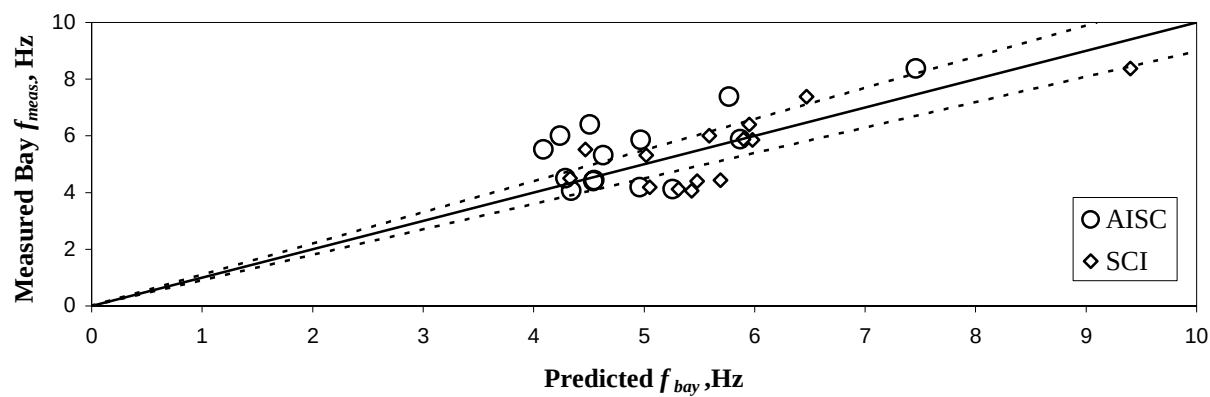
Building	File	Bay	Predicted Frequencies (AISC)					Meas. f_m (Hz)	Frequency Ratios		
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_{bay}/f_m	f_n^4/f_m	f_n^6/f_m
Cardington	BR1	1	5.82	9.17	4.97	5.61	5.76	5.86	0.848	1.008	1.037
Le Colisee	BR2	1	5.26	N/A	5.26	5.26	5.26	4.13	1.275	1.275	1.275
		2	4.96	N/A	4.96	4.96	4.96	4.19	1.184	1.184	1.184
		3	4.55	N/A	4.55	4.55	4.55	4.44	1.025	1.025	1.025
		4	4.34	N/A	4.34	4.34	4.34	4.06	1.068	1.068	1.068
		5	4.29	N/A	4.29	4.29	4.29	4.50	0.953	0.953	0.953
SCI HQ	BR3	1	11.56	9.77	7.46	8.81	9.28	8.38	0.890	1.052	1.107
Appold St.	BR4	1	5.87	N/A	5.87	5.87	5.87	5.88	0.998	0.998	0.998
Finsbury	BR5	1	6.38	6.38	4.51	5.36	5.68	6.40	0.705	0.838	0.888
		2	5.98	6.00	4.23	5.04	5.34	6.00	0.705	0.839	0.891
Grosvenor	BR6	1	6.04	6.89	4.54	5.38	5.67	4.40	1.032	1.222	1.290
Lloyds	BR7	1	9.85	5.24	4.63	5.14	5.22	5.32	0.870	0.966	0.981
		2	8.53	4.66	4.09	4.56	4.64	5.52	0.741	0.826	0.841
West Bldg.	BR8	1	6.27	12.51	5.77	6.17	6.25	7.38	0.782	0.837	0.847

Average -	0.934	1.003	1.024
Standard Deviation -	0.175	0.147	0.147

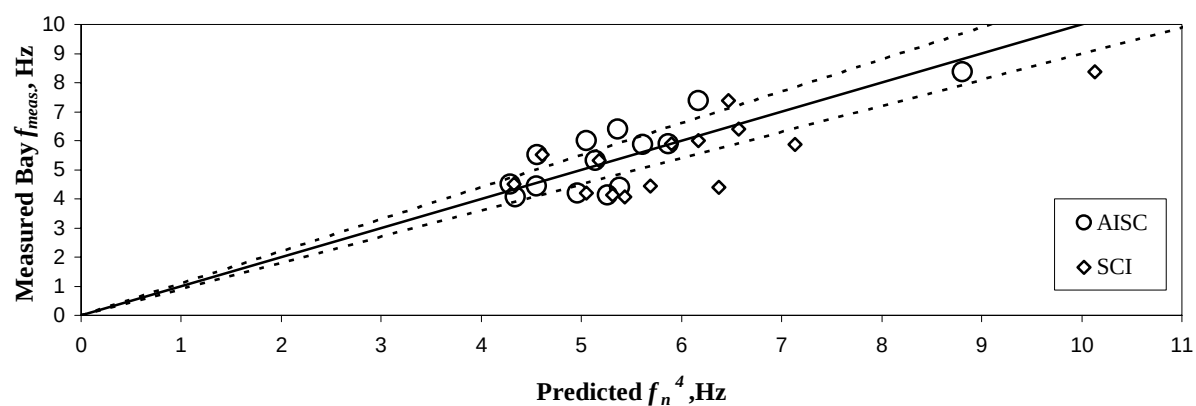
Table 2.14 – System Frequency Comparison for British Floors Using SCI Method

Building	File	Bay	Predicted Frequencies (SCI)					Meas. f_m (Hz)	Frequency Ratios			$f_{bay}^{AISC} / f_{bay}^{SCI}$
			f_b (Hz)	f_g (Hz)	f_{bay} (Hz)	f_n^4 (Hz)	f_n^6 (Hz)		f_{bay}/f_m	f_n^4/f_m	f_n^6/f_m	
Cardington	BR1	1	9.03	8.07	5.98	7.13	7.53	5.86	1.020	1.217	1.286	0.831
Le Colisee	BR2	1	5.31	N/A	5.31	5.31	5.31	4.13	1.287	1.287	1.287	0.991
		2	5.05	N/A	5.05	5.05	5.05	4.19	1.206	1.206	1.206	0.982
		3	5.69	N/A	5.69	5.69	5.69	4.44	1.282	1.282	1.282	0.800
		4	5.43	N/A	5.43	5.43	5.43	4.06	1.337	1.337	1.337	0.799
		5	4.33	N/A	4.33	4.33	4.33	4.50	0.962	0.962	0.962	0.991
SCI HQ	BR3	1	25.45	10.19	9.40	10.13	10.18	8.38	1.122	1.208	1.215	0.794
Appold St.	BR4	1	5.90	N/A	5.90	5.90	5.90	5.88	1.003	1.003	1.003	0.995
Finsbury	BR5	1	13.60	6.67	5.95	6.57	6.65	6.40	0.930	1.027	1.039	0.758
		2	12.74	6.26	5.59	6.17	6.25	6.00	0.932	1.029	1.041	0.757
Grosvenor	BR6	1	9.53	6.73	5.48	6.37	6.60	4.40	1.245	1.447	1.500	0.828
Lloyds	BR7	1	21.43	5.18	5.02	5.18	5.18	5.32	0.944	0.973	0.974	0.922
		2	18.56	4.61	4.47	4.61	4.61	5.52	0.810	0.834	0.835	0.915
West Bldg.	BR8	1	6.47	N/A	6.47	6.47	6.47	7.38	0.877	0.877	0.877	0.892

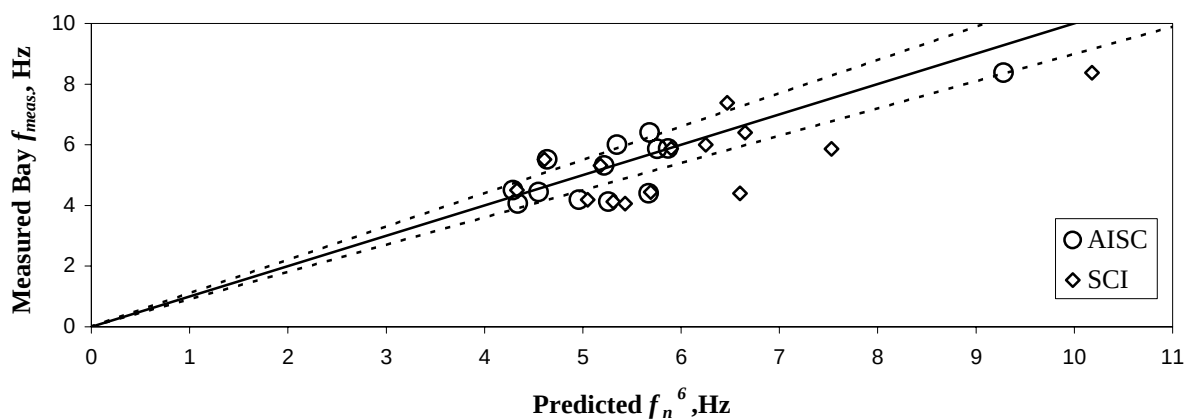
Average -	1.068	1.121	1.132	0.875
Standard Deviation -	0.174	0.186	0.196	0.091



(a) Measured Bay Frequency vs. Predicted Bay Frequency (f_{bay})



(b) Measured Bay Frequency vs. Predicted Bay Frequency (f_n^4)



(c) Measured Bay Frequency vs. Predicted Bay Frequency (f_n^6)

Figure 2.11 – Measured Bay Frequency vs. Predicted AISC and SCI Bay Frequency for British Tested Floors

The predicted bay frequencies, f_n^2 , are plotted against the measured system frequencies in Figure 2.11(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only 6 of the 14 floors studied by SCI but using the AISC prediction method. The predicted bay frequencies, using the revision f_n^4 , are plotted against the measured system frequencies in Figure 2.11(b), where the predicted system frequencies are within 10 % of the measured bay frequencies for 7 of the 14 floors. The predicted bay frequencies, using the revision f_n^6 , are plotted against the measured frequencies in Figure 2.11(c), where the predicted system frequencies are within 10 % of the measured bay frequencies for 7 of the 14 floors studied by SCI.

2.5.2 SCI FREQUENCY EVALUATION OF BRITISH FLOORS

The methods provided by Steel Construction Institute also proved to be a fairly accurate tool for predicting the fundamental natural frequencies of the British floor systems. From Table 2.14, the average ratio of predicted system frequency to measured frequency (f_{bay}/f_m) is 1.07. However, a modest standard deviation of 0.17 suggests that the data is moderately scattered. The average ratio of predicted system frequency to measured frequency (f_n^4/f_m) is 1.12 with a standard deviation of 0.19, and for (f_n^6/f_m) is 1.13 with a standard deviation of 0.20.

The predicted bay frequencies, f_n^2 , are plotted against the measured system frequencies in Figure 2.11(a). This figure illustrates that the predicted bay frequencies are within 10 % of the measured bay frequencies for only 6 of the 14 floors studied by SCI. The predicted bay frequencies, using the revision f_n^4 , are plotted against the measured system frequencies in Figure 2.11(b). This figure illustrates that the predicted system frequencies are within 10 % of the measured bay frequencies for only 5 of the 14 British case studies. The predicted bay frequencies, using the revision f_n^6 , are plotted against the measured frequencies in Figure 2.11(c). This figure illustrates that the predicted system frequencies are within 10 % of the measured bay frequencies for 5 of 14 British case studies.

2.5.3 EXAMPLE EVALUATION OF A SCI TESTED FLOOR

To further explore the differences between the AISC and SCI methods, basic system properties are investigated and compared in this section. Specifically, differences in effective width, composite moment of inertia, and frequency are examined for a particular floor tested by SCI. In Section 2.5.2, the SCI method assumes a fixed-ended boundary condition for the secondary framing when evaluating a floor based on the flexibility of the primary framing. However, for purposes of comparison, simply supported conditions are assumed for both beam and girder frequency calculations in this section. Also, reductions of deflections due to stiffening provided by adjacent bays are neglected for both methods.

The floor system used in the comparison, shown in Figure 2.11, is of the SCI headquarters in Ascot, Great Britain. The secondary floor framing of a typical bay consists of 305X127UB42 beams spaced at 98 in. with a span of 19 ft-8in. The beams are supported by 686X152 castellated girders. The girders span 24 ft-5 in. The floor slab is 5-1/8 in. total depth normal weight concrete supported by 2 in. deep composite form deck. The loading on the bay includes the self-weight of the slab and decking as well as non-structural components such as services, ceilings, raised floors, mechanical ductwork, and office furniture.

Table 2.15 shows the general, beam mode and girder mode properties for the AISC and SCI methods. The concrete compressive strength (f_c') is not used in the United Kingdom. The equivalent concrete compressive strength used in AISC calculations is found by back calculating from the dynamic modular ratio of 5.29 used by SCI for normal weight concrete. The dynamic modular ratio is taken as the modulus of elasticity for steel (E_s) divided by 1.35 times the modulus of elasticity for concrete (E_c). To account for dynamic effects under load, the modulus of elasticity for concrete is increased. The resulting equivalent concrete compressive strength used in the AISC calculations is 5.0 ksi. (Note : For lightweight concrete, the dynamic modular ratio used by SCI is 9.31 and the corresponding concrete compressive strength is 3.8 ksi.)

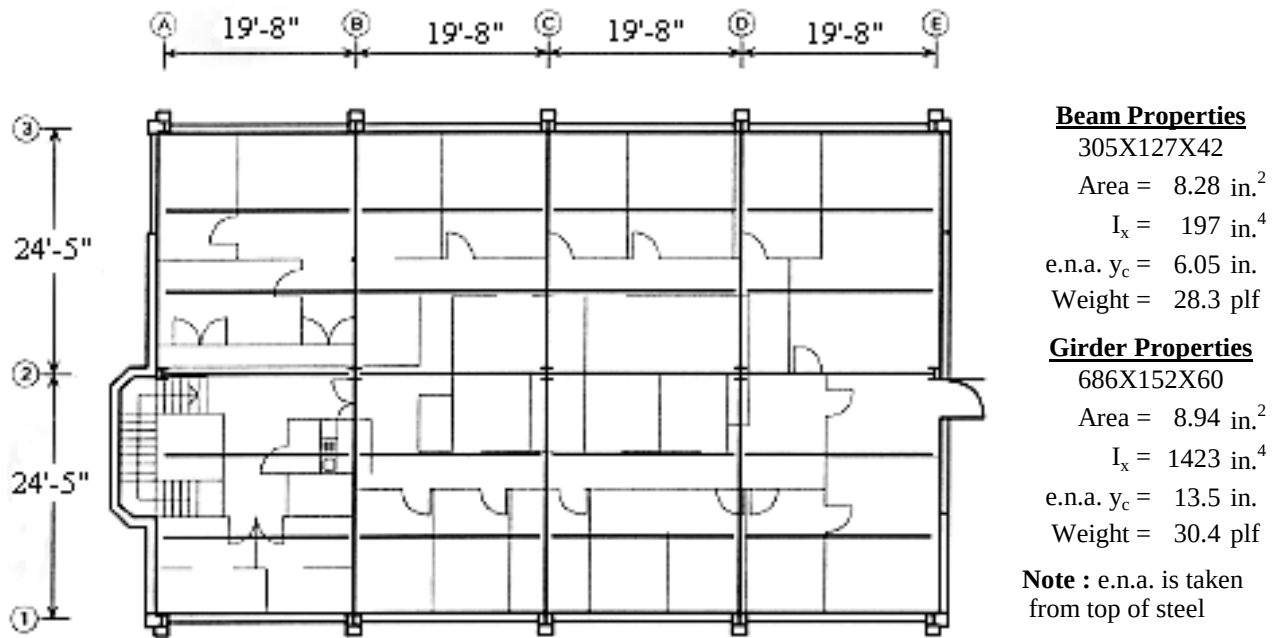


Figure 2.12 – First Floor Framing Plan SCI Headquarters (Hicks, et al. 2000)

Table 2.15 – AISC vs. SCI Comparison of Parameters for the SCI Headquarters Framing

		AISC	SCI
General Properties	f_c (ksi) -	5.0	N/A
	E_c (ksi) -	3985	5510
	E_s (ksi) -	29000	29725
	Modular Ratio n -	5.39	5.39
	Total Weight (psf) -	85.9	85.9
Beam Mode Properties	b_{eff} (ft) -	8.15	4.92
	I_{comp} (in. ⁴) -	910.6	840.7
	w_j (lb./ft) -	727.7	727.7
	Δ_b (in.) -	0.094	0.101
	f_b (Hz) -	11.56	11.14
Girder Mode Properties	b_{eff} (ft) -	9.78	6.11
	I_{comp} (in. ⁴) -	3778.3	3604.9
	w_g (lb./ft) -	1788.9	1788.9
	Δ_g (in.) -	0.131	0.137
	f_g (Hz) -	9.77	9.54
Combined	$f_n^2 = f_{bay}$ (Hz) -	7.46	7.25

From Table 2.15, the effective width of the concrete slab for the secondary beams is the member spacing of 8.15 ft according to the AISC Design Guide. In contrast, SCI estimates the effective width of slab to be much smaller, 4.92 ft. Subsequently in the SCI method, the composite moment of inertia (I_{comp}) is reduced and the member deflection (Δ_b) increases. The final deflections based on the AISC and SCI techniques are 0.094 in. and 0.101 in., respectively. The corresponding natural frequencies are 11.56 Hz and 11.14 Hz for the AISC and SCI methods, respectively.

The effective width of the concrete slab for the primary beams is 40 % of the girder span or 9.78 ft according to the AISC Design Guide. In contrast, SCI again estimates the effective width of concrete slab for the girders to be much smaller with a value of 25 % of the girder length or 6.11 ft. Subsequently in the SCI method, the composite moment of inertia (I_{comp}) is reduced and the member deflection (Δ_g) increases. The final deflections based on AISC and SCI techniques are 0.131 in. and 0.137 in., respectively, and the corresponding natural frequencies are 9.77 Hz and 9.54 Hz. The bay frequency using Dunkerly's estimation is 7.46 Hz based on the AISC Design Guide and 7.25 Hz based on the SCI method. Example frequency calculations using both methods are found in Appendix A to this report.

2.6 SUMMARY

The primary goal of this part of this floor vibrations study was to compare predicted and measured frequencies for floor systems currently occupied and under construction in the United States and Europe. This comparison was compiled in an attempt to improve predictions of floor response due to occupant-induced excitations. Floor system properties and framing for 103 case studies were scrutinized using AISC and SCI methods to find correlations between predicted and measured frequencies. Table 2.16 summarizes the major results of the frequency studies.

Hot-Rolled Framing. For structural floor systems with hot-rolled framing, the comparison of predicted and measured floor frequencies shows that the system frequency, f_n^2 , estimated using Equation 1.12 and the AISC Design Guide, is not a reliable prediction of bay frequency (avg. $f_n^2/f_m = 0.797$ with $\sigma = 0.132$). In many of these case studies, floor motion is limited to the beam motion, because the girders have

significant rigidity and do not affect the system frequencies. This comparison shows that the predicted beam frequency calculated using the Design Guide is more indicative of the actual bay frequency than Dunkerly's estimate of f_n^2 . In addition, the modifications to Dunkerly's equation, f_n^4 and f_n^6 , only marginally improve the estimate of the actual system frequency than predictions using f_n^2 for floors with hot-rolled framing. However, the scatter is quite significant for these modified predictions. Floors with hot-rolled framing did not have significant correlations between predicted girder and measured system frequencies.

Table 2.16 – Summary of Results of Frequency Study

	AISC Design Guide 11						SCI 076					
Type of Framing	H-R	H-R w/J	J-G w/J	H-R w/Cast	BR	Over-All	H-R	H-R w/J	J-G w/J	H-R w/Cast	BR	Over-All
Avg. f_{beam}/f_m Std.Dev. σ	1.034 0.176	1.059 0.252	1.003 0.225	1.058 0.078	N/A	1.038 0.225	N/A	N/A	N/A	N/A	N/A	N/A
Avg. f_n^2/f_m Std.Dev. σ	0.797 0.132	0.896 0.196	0.759 0.166	0.858 0.090	0.934 0.18	0.839 0.176	0.961 0.196	0.981 0.217	0.783 0.170	0.992 0.026	1.068 0.174	0.946 0.205
Avg. f_n^4/f_m Std.Dev. σ	0.897 0.151	0.979 0.214	0.842 0.172	0.965 0.050	1.003 0.147	0.926 0.182	0.991 0.190	0.998 0.212	0.819 0.181	1.044 0.045	1.121 0.186	0.976 0.206
Avg. f_n^6/f_m Std.Dev. σ	0.931 0.158	1.004 0.218	0.873 0.181	1.002 0.046	1.024 0.147	0.955 0.186	0.996 0.189	0.999 0.212	0.822 0.182	1.050 0.050	1.132 0.196	0.980 0.209
* Floors where $f_{beam} = f_{meas.}$ (%)	64	40	39	80	N/A	43	N/A	N/A	N/A	N/A	N/A	N/A
* Floors where $f_n^2 = f_{meas.}$ (%)	28	30	18	40	43	29	32	33	29	100	43	38
* Floors where $f_n^4 = f_{meas.}$ (%)	40	33	29	100	50	38	48	33	36	80	36	41
* Floors where $f_n^6 = f_{meas.}$ (%)	28	40	32	100	50	38	48	33	36	80	36	41

Notes: “H-R” stands for hot-rolled framing, “H-R/J” represents hot-rolled girders and joist framing, “H-R/Cast” represents hot-rolled girders and castellated framing, “J-G/J” represents joist girders with joists, and “BR” represents British floors assessed in Section 2.5.

* These rows measure the percentage of case studies that have predicted beam/bay frequencies within a tolerance of $\pm 10\%$ of the measured frequency.

The methods used by the Steel Construction Institute Guide are reliable for predicting bay frequencies, f_n^2 , for floors constructed of hot-rolled shapes (avg. $f_n^2/f_m = 0.961$ with $\sigma = 0.196$). The modifications to Dunkerly's equation, f_n^4 and f_n^6 , also provide better estimates of the system frequency than predictions using f_n^2 for floors with hot-rolled framing. However, the scatter is quite significant for these modified predictions. Floors with hot-rolled framing evaluated using SCI techniques did not have significant correlations between predicted beam or girder and measured system frequencies.

Hot-Rolled Girders and Joist Beams. For structural floor systems with hot-rolled girders and joist framing, the comparison of predicted and measured floor frequencies reveals that the system frequency, f_n^2 , estimated using Equation 1.12 and the AISC Design Guide, is again not a reliable prediction of bay frequency (avg. $f_n^2/f_m = 0.896$ with $\sigma = 0.196$). This comparison also shows that the predicted beam frequency is not an accurate prediction of the fundamental natural frequency of floor systems utilizing hot-rolled and joist framing. The modifications to Dunkerly's equation, f_n^4 and f_n^6 , estimate the system frequency with the same accuracy as predictions using f_{beam} . However, the scatter is quite significant for these modified predictions.

The methods used by the Steel Construction Institute Guide are reliable for predicting bay frequencies using Dunkerly's approximation, f_n^2 , for floors constructed of hot-rolled girders and joists (avg. $f_n^2/f_m = 0.981$ with $\sigma = 0.217$). In addition, the modifications to Dunkerly's equation, f_n^4 and f_n^6 , estimate the system frequency with better accuracy than predictions using f_n^2 for floors with hot-rolled girders and joist framing. However, the scatter is quite significant for these modified predictions. Floors with hot-rolled girders and joist framing evaluated using SCI techniques did not have significant correlations between predicted beam or girder and measured system frequencies.

Joist Girders and Joists. For structural floor systems with joist girders and joists, the comparison of predicted and measured floor frequencies shows that the system frequency, f_n^2 , estimated using Equation 1.12 and the AISC Design Guide, is not a reliable prediction of bay frequency (avg. $f_n^2/f_m = 0.759$ with $\sigma = 0.166$). This comparison also shows that the predicted joist frequency is not an accurate prediction of the fundamental natural frequency of floor systems utilizing joist girder and joist framing.

The modifications to Dunkerly's equation, f_n^4 and f_n^6 , estimate the system frequency with less accuracy than predictions using f_n^2 . The scatter is quite significant for these modified predictions.

The methods used by the Steel Construction Institute Guide are also not reliable for predicting bay frequencies, f_n^2 , of floors constructed with joist girders and joists (avg. $f_n^2/f_m = 0.783$ with $\sigma = 0.170$). In addition, the modifications to Dunkerly's equation, f_n^4 and f_n^6 , marginally improve the estimate of the system frequency than predictions using f_n^2 for floors with joist girders and joists. However, the scatter is quite significant for these modified predictions. Floors with joist girders and joist framing evaluated using SCI techniques did not have significant correlations between predicted beam or girder and measured system frequencies.

Hot-Rolled Girders and Castellated Beams. Only six case studies of bays with hot-rolled girders and castellated framing were evaluated in the study. For structural floor systems with hot-rolled girders and castellated beams, the comparison of predicted and measured floor frequencies shows that the system frequency, f_n^2 , estimated using Equation 1.12 and the AISC Design Guide, is reliable for about 50 % of the predictions (avg. $f_n^2/f_m = 0.858$ with $\sigma = 0.090$). This comparison shows that the predicted beam frequency is more indicative of the bay frequency than Dunkerly's estimate of f_n^2 . In addition, the modifications to Dunkerly's equation, f_n^4 and f_n^6 , estimate the system frequency with better accuracy than predictions using f_n^2 for floors with hot-rolled girders and castellated framing. In addition, the scatter in the data is quite low for these modified predictions.

The methods used by the Steel Construction Institute Guide are reliable for predicting bay frequencies, f_n^2 , for floors constructed of hot-rolled girders and castellated framing (avg. $f_n^2/f_m = 0.992$ with $\sigma = 0.026$). The modifications to Dunkerly's equation, f_n^4 and f_n^6 , also estimate the system frequency with good accuracy for floors with hot-rolled and castellated framing. However, the scatter is quite significant for these modified predictions. Floors with hot-rolled girders and castellated framing evaluated using SCI techniques did not have significant correlations between predicted beam or girder and measured system frequencies.

British Floors. The AISC Design Guide on average underestimated the bay frequency by 7 % for the floor systems previously evaluated in the SCI publication *Design Guide for Vibration of Long Span Composite Floors* (Hicks, et al. 2000). The SCI method on average overestimated the bay frequency by 7 % for the same floor systems. Modest scatter in the data suggests that the predicted bay frequency, f_n^2 , calculated using both methods is a moderately reliable prediction for these tested floors. The differences between the predicted frequencies arise from varying assumptions on deflection reductions and boundary conditions for deflection estimations. The AISC Design Guide and SCI also have different definitions for the effective width of slab, which has significantly affects the estimation of the composite moment of inertia for primary and secondary beams.

CHAPTER III – FLOOR VIBRATIONS CRITERIA COMPARISONS

3.1 INTRODUCTION

Predicting the response of framing systems is important when evaluating floors for acceptability. Many criteria are used in the United States and the United Kingdom to help determine the tolerability of floor system response due to occupant-induced vibrations. This chapter evaluates occupied floor systems for case studies using four separate acceptability criteria and compares the results with subjective analyses based on field observations. The four acceptability criteria used to evaluate these case studies in this chapter are the American Institute of Steel Construction Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997), the Modified Reiher-Meister scale, the Murray Criterion, and enhancements to the Steel Construction Institute criteria outlined in *Design Guide for Vibrations of Long Span Composite Floors* (Hicks, et al. 2000). Details of the backgrounds of these four acceptability criteria are discussed in Chapter I of this study. The AISC and SCI acceptability criteria are the most current methods used for evaluating floor system behavior in the United States and Great Britain. With the advent of light-weight and stronger construction materials, past assumptions and methods required for the Modified Reiher-Meister scale and the Murray Criterion are becoming outdated. Therefore, comparing past and present acceptability criteria will provide insight into the development and progress of floor vibration research. Through comparison of analyses and experimental data for existing floor systems, the correlation between predicted and measured behavior may be improved.

This chapter provides a comparison of predicted and observed behavior from 51 case studies involving office floors currently being occupied and used in the United States. The type of framing used during construction differentiates the structural floor systems. Similar to the previous chapter, the classifications for framing are “hot-rolled beams”, “hot-rolled beams with joists”, and “joist girders with joists”. The controlling girder, which has the most deflection and the lowest frequency, is considered when

classifying a floor system with two different types of girders used in the framing. (The castellated framing case studies were not fully constructed at the time of testing. Therefore, these case studies are not evaluated in this chapter.) The results from the acceptability evaluations are then compared with subjective field observations of actual floor behavior and system properties at the time of the testing. The subjective analyses are based on the expertise of the measurement team present, as well as, the opinions of the occupants who were occupying the office spaces at the time of testing.

Supplemental material for this report is found in the publication by Boice and Murray (2003) and contains summary sheets of the case studies used in the frequency and criteria comparisons. These sheets include floor systems properties such as damping, framing, loading, predicted and measured fundamental frequencies, accelerations, and slab properties. In addition, the supplemental material contains framing plans for the floor systems evaluated in this chapter. Appendix A provides example calculations for determining floor acceptability based on the Steel Construction Institute criteria. The sample calculations for SCI accelerations and acceptability are based on predicted SCI system frequencies.

3.2 CRITERIA COMPARISON RESULTS

Tables 3.1 through 3.3 summarize the results from the acceptability evaluations for the assessed case studies. Agreement between subjective analyses and acceptability criteria presented in the Design Guide is based on the predicted peak accelerations of the office floor systems. For example, floors with predicted peak accelerations above 0.50 % of gravity should exhibit annoying floor vibrations and subsequently have complaints from the occupants. Agreement between subjective analyses and the Murray Criterion is based on a minimum required damping value needed to limit vibrations of office floor systems. For the Murray Criterion, building owners with floor systems having required damping value above 4.5 % should receive complaints from the occupants due to the vibrations. Agreement between subjective analyses and the Modified Reiher-Meister scale is based on the chart shown in Figure 1.6. According to the figure, floor systems that plot above the upper one-half of the distinctly perceptible range will result in complaints from the occupants (Murray 1975). Agreement between subjective analyses

and acceptability criteria presented by the Steel Construction Institute is based on the root-mean-square (rms) accelerations of the office floor systems. For example, floors with root-mean-square accelerations above the acceleration limits should also exhibit annoying floor vibrations and subsequently have complaints from the occupants. The acceleration limits are based on the type of office environment and established using Equations 1.21 and 1.22. In addition, the damping value, β , corresponding to recommendations in the AISC Design Guide for the location at the time of testing, as well as, predicted and measured system frequencies are provided in Tables 3.1 through 3.3 in order to draw a parallel with the results of the criteria comparisons.

3.2.1 FLOORS WITH HOT-ROLLED SHAPES

Table 3.1 shows the results of the criteria comparisons for floor systems comprised of hot-rolled shapes for primary members. From the data presented, 17 of 19 or about 89 % of the case studies analyzed using the Design Guide criteria agree with the corresponding subjective analyses of the behavior of the floor systems. When the case studies were evaluated using the Murray Criterion, about 68 % of the floor systems comply with the corresponding subjective analyses. The case studies evaluated using the Modified Reiher-Meister acceptability scale have the least compliance with only 47 % of the floor systems agreeing with the subjective analyses of the system performance. The case studies evaluated using the Steel Construction Institute acceptability criteria have about 68 % compliance with the subjective evaluations. Only about 42 % of the case studies have complete agreement between all four of the acceptability criteria and the corresponding subjective analyses. Only about 11 % of the case studies show complete disagreement between all four of the acceptability criteria and the corresponding subjective analyses.

3.2.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS

Table 3.2 shows the results of the criteria comparisons for floor systems comprised of hot-rolled shapes for primary members and joist framing for secondary members. From the data presented, 13 of 15 or about 87 % of the case studies analyzed using the Design Guide criteria agree with the corresponding subjective analyses of the behavior of the floor systems. When the case studies were evaluated using the Murray

Table 3.1 – Acceptability Criteria Comparison for Floors with Hot-Rolled Shapes

Building	Bay	β (%)	Pred. f_{bay} (Hz)		$f_{meas.}$ (Hz)	Subjective Analysis	D.G. 11 a_{peak}/g (%)	Murray Criterion Rqd. Damping (%)		Modified Reiher-Meister	SCI		
			D.G. 11	SCI 076							a_{peak}/g (%)	a_{rms}/g (%)	a_{limit}/g (%)
The Jersey Lil Club ISU	1	3.0	3.15	3.23	4.93	N-C	0.34	3.2	✓	N-P	0.80	0.57	0.41
	1	2.5	3.19	4.07	3.88	M-C	1.08	4.1	✓	S-P	0.53	0.37	0.41
	2,3	2.5	3.82	4.20	3.75	M-C	0.71	4.2	✓	S-P	0.54	0.38	0.41
	4	2.5	3.82	4.12	4.00	M-C	0.71	4.2	✓	S-P	0.54	0.38	0.41
Newgen Building	Ext.	3.0	4.64	5.60	6.50	N-C	0.52	4.4	✓	A/D-P	0.55	0.39	0.41
Penn State	1	3.0	4.23	5.68	4.90	N-C	0.42	3.7	✓	S-P	0.58	0.41	0.41
	1,2	2.5	3.81	4.89	5.25	C	0.69	4.0	✓	S-P	0.39	0.28	0.20
	3,4	2.5	3.81	4.89	4.13	C	0.69	4.0	✓	S-P	0.39	0.28	0.20
Oracle Building	5	2.5	3.88	4.93	4.50	C	0.73	4.2	✓	S-P	0.38	0.27	0.20
	Typ.	2.5	3.38	3.84	4.00	N-C	0.46	3.4	✓	S-P	0.34	0.24	0.41
Newport V	Typ.	2.5	4.69	5.73	7.00	N-C	0.31	3.6	✓	S-P	0.25	0.18	0.41
Doylestown Hosp.	Typ.	3.0	3.29	3.96	N/A	C	0.40	3.4	✓	S-P	0.25	0.18	0.41
Stanley D. Lindsey	Typ.	2.5	4.24	4.24	5.25	C	0.56	3.5	✓	S-P	0.71	0.50	0.41
AMEX	Typ.	3.0	4.00	5.08	5.50	M-C	0.43	3.8	✓	S-P	0.44	0.31	0.41
GTE	Typ.	3.0	4.20	5.70	4.25	M-C	0.59	4.4	✓	A/D-P	0.68	0.48	0.41
Three Parkwood	1	3.0	3.65	4.87	4.00	M-C	0.75	4.0	✓	S-P	0.89	0.63	0.41
	2	3.0	4.35	5.32	5.00	N-C	0.38	3.8	✓	S-P	0.43	0.30	0.41
Norman Pointe 1	3	3.0	3.38	4.68	5.25	N-C	0.37	3.7	✓	S-P	0.20	0.14	0.41
	4	3.0	4.35	5.32	5.25	N-C	0.38	3.8	✓	S-P	0.43	0.30	0.41

Notes :

1. The gray shading in the Design Guide 11 column indicates that the a_{peak}/g of the floor system is less than or equal to 0.50 % of gravity and is an acceptable floor.
2. The gray shading in the SCI column indicates that the a_{rms}/g of the floor system is less than or equal to the corresponding a_{limit}/g and is an acceptable floor.
3. The check marks indicate that the particular criterion complies with the corresponding subjective analysis. The “X” marks indicate that the particular criterion does not comply with the corresponding subjective analysis.
4. In the Subjective Analysis column, “N-C” represents no complaints, “C” represents complaints, and “M-C” represents many complaints.
5. In the Modified Reiher-Meister column, “N-P” represents not perceptible, “S-P” represents slightly perceptible, “A/D-P” represents acceptable but distinctly perceptible, “U/D-P” represents unacceptable and distinctly perceptible.

Table 3.2 – Acceptability Criteria Comparison for Floors with Hot-Rolled Girders and Joists

Building	Bay	β (%)	Pred. f_{bay} (Hz)		$f_{meas.}$ (Hz)	Subjective Analysis	D.G. 11 a_{peak}/g (%)	Murray Criterion Reqd. Damping (%)	Modified Reiher-Meister	SCI		
			D.G. 11	SCI 076						a_{peak}/g (%)	a_{rms}/g (%)	a_{limit}/g (%)
ACTIC Building Camron Univ.	1	3.0	5.55	5.55	6.50	N-C	0.29	✓	S-P	0.57	0.40	0.41
	Rm.271	3.0	5.65	6.16	7.00	N-C	0.35	✓	S-P	0.68	0.48	0.41
	Rm.273	3.0	5.39	5.88	6.75	N-C	0.35	✓	S-P	0.62	0.44	0.41
Melroe Corp. Head	1	3.0	3.80	5.46	5.50	M-C	0.74	✓	A/D-P	0.86	0.61	0.41
Steel Case	Mezz.	2.5	3.59	4.45	4.75	C	0.89	✓	S-P	0.55	0.39	0.41
	Int.	2.5	3.19	3.46	4.75	N-C	0.51	✓	S-P	2.27	1.61	0.41
Patterson Ave Fac.	4	3.0	5.97	6.58	4.50	N-C	0.42	✓	A/D-P	0.47	0.33	0.41
	5	3.0	5.75	6.58	4.50	C	0.45	✓	A/D-P	0.48	0.34	0.41
	7	3.0	4.68	5.47	4.50	C	0.62	✓	A/D-P	0.45	0.32	0.41
InterQuest/ Ford	1	2.5	3.97	5.04	4.75	C	0.73	✓	S-P	0.90	0.64	0.41
XILINX	1	2.5	3.93	5.05	4.75	M-C	0.61	✓	S-P	0.78	0.55	0.41
Civic Center	1 (ltd.)	2.5	5.16	5.16	5.25	M-C	0.73	✓	A/D-P	1.28	0.91	0.41
	2 (uldd.)	2.5	5.26	5.26	5.25	M-C	0.73	✓	A/D-P	1.32	0.93	0.41
Gateway 800 Bridge	Typ.	3.0	2.22	2.52	4.00	N-C	0.47	✓	N-P	1.20	0.85	0.41
MidWest Kentworth	Typ.	2.5	4.04	4.04	5.75	N-C	0.57	✓	S-P	0.58	0.41	0.41

Notes :

1. The gray shading in the Design Guide 11 column indicates that the a_{peak}/g of the floor system is less than or equal to 0.50 % of gravity and is an acceptable floor.
2. The gray shading in the SCI column indicates that the a_{rms}/g of the floor system is less than or equal to the corresponding a_{limit}/g and is an acceptable floor.
3. The check marks indicate that the particular criterion complies with the corresponding subjective analysis. The “X” marks indicate that the particular criterion does not comply with the corresponding subjective analysis.
4. In the Subjective Analysis column, “N-C” represents no complaints, “C” represents complaints, and “M-C” represents many complaints.
5. In the Modified Reiher-Meister column, “N-P” represents not perceptible, “S-P” represents slightly perceptible, “A/D-P” represents acceptable but distinctly perceptible, “U/D-P” represents unacceptable and distinctly perceptible.

Table 3.3 – Acceptability Criteria Comparison for Floors with Joist Girders and Joist Framing

Building	Bay	β (%)	Pred. f_{bay} (Hz)		f_{meas} (Hz)	Subjective Analysis	D.G. 11 a_{peak}/g (%)	Murray Criterion Rqd. Damping (%)		Modified Reiher-Meister	SCI		
			D.G. 11	SCI 076							a_{peak}/g (%)	a_{rms}/g (%)	a_{limit}/g (%)
Patterson Ave Fac.	6	3.0	4.67	4.72	5.00	N-C	0.53	X	3.8	✓	1.04	0.74	0.41
Toyota Motor Corp	1	2.0	4.53	4.94	5.63	M-C	0.66	✓	3.9	X	0.86	0.61	0.20
Sony Corp Building	1	2.5	3.45	3.83	5.50	M-C	0.95	✓	4.9	✓	1.15	0.81	0.20
Diversa Building	Ext.	3.0	3.43	4.02	5.00	M-C	1.12	✓	4.3	X	0.76	0.54	0.41
Pacific Tech	Typ.	3.0	3.74	4.44	6.00	M-C	0.77	✓	4.3	X	0.87	0.62	0.41
Garmin Int. Bld.	Int./Ext.	2.5	3.66	3.94	6.25	C	1.08	✓	5.8	✓	1.45	1.03	0.20
IBM	1	2.5	6.78	6.78	6.00	C	0.54	✓	4.4	X	1.09	0.77	0.20
Southern Bell	2	3.0	3.91	4.51	6.25	N-C	0.48	✓	3.7	✓	0.70	0.49	0.41
	5,7	3.0	3.91	4.51	6.75	N-C	0.48	✓	3.7	✓	0.70	0.49	0.41
Wholesale Grocers	9	3.0	4.54	4.77	9.50	N-C	0.54	X	4.5	✓	1.04	0.74	0.41
	1	3.0	2.45	2.45	3.50	N-C	0.21	✓	2.7	✓	0.57	0.40	0.41
	3	3.0	2.53	2.53	4.25	N-C	0.21	✓	2.7	✓	0.57	0.40	0.41
R.S.I. Bldg.	5	3.0	2.03	2.03	3.00	N-C	0.22	✓	2.8	✓	0.51	0.36	0.41
	1	2.0	8.54	8.54	9.50	C	0.97	✓	7.6	✓	5.57	3.94	0.44
	2	2.0	8.54	8.54	9.75	C	0.97	✓	7.6	✓	5.57	3.94	0.44
	3	3.0	9.61	9.61	9.75	C	0.68	✓	6.3	✓	3.08	2.18	0.49
	4	5.0	6.87	6.87	7.25	C	0.54	✓	6.3	✓	1.56	1.10	0.41

Notes :

1. The gray shading in the Design Guide 11 column indicates that the a_{peak}/g of the floor system is less than or equal to 0.50 % of gravity and is an acceptable floor.
2. The gray shading in the SCI column indicates that the a_{rms}/g of the floor system is less than or equal to the corresponding a_{limit}/g and is an acceptable floor.
3. The check marks indicate that the particular criterion complies with the corresponding subjective analysis. The “X” marks indicate that the particular criterion does not comply with the corresponding subjective analysis.
4. In the Subjective Analysis column, “N-C” represents no complaints, “C” represents complaints, and “M-C” represents many complaints.
5. In the Modified Reiher-Meister column, “N-P” represents not perceptible, “S-P” represents slightly perceptible, “A/D-P” represents acceptable but distinctly perceptible, “U/D-P” represents unacceptable and distinctly perceptible.

Criterion, about 67 % of the floor systems comply with the corresponding subjective analyses. The case studies evaluated using the Modified Reiher-Meister acceptability scale have the least compliance with only 47 % of the floor systems agreeing with the subjective analyses. The case studies evaluated using the Steel Construction Institute acceptability criteria have about 53 % compliance with the subjective evaluations. For about 7 % of the case studies, all four of the acceptability criteria and the corresponding subjective analyses have complete agreement. None of the case studies have complete disagreement between all four of the acceptability criteria and the corresponding subjective analyses.

3.2.3 FLOORS WITH JOIST GIRDERS AND JOISTS

Table 3.3 shows the results of the criteria comparisons for floor systems comprised of joist girders for primary members and joist framing for secondary members. From the data presented, 15 of 17 or about 88 % of the case studies analyzed using the Design Guide criteria agree with the corresponding subjective analyses of the behavior of the floor systems. When the case studies were evaluated using the Murray Criterion, about 71 % of the floor systems comply with the corresponding subjective analyses. The case studies evaluated using the Modified Reiher-Meister acceptability scale have the least compliance with only 65 % of the floor systems agreeing with the subjective analyses. The case studies evaluated using the Steel Construction Institute acceptability criteria have about 82 % compliance with the subjective evaluations. For about 41 % of the case studies, all four of the acceptability criteria and the corresponding subjective analyses have complete agreement. None of the case studies have complete disagreement between all four of the acceptability criteria and the corresponding subjective analyses.

3.3 SUMMARY

The primary goal of this part of the study was to determine which floor vibrations criteria has the best accuracy for predicting floor response. Table 3.4 summarizes the major results from the acceptability criteria comparisons.

Table 3.4 – Criteria Comparison Summary

Framing	Agreement: Design Guide 11 (%)	Agreement: Modified Reiher-Meister (%)	Agreement: Murray Criterion (%)	Agreement: SCI Criteria (%)
Hot-Rolled Shapes	89	47	68	68
Hot-Rolled w/Joists	87	47	67	53
Joist Girders w/Joists	88	65	71	82

Note: Agreement refers to the percentage of cases studies where the subjective analyses complies with the specific criteria.

From this study, the methods and criteria established in the AISC Design Guide give the best results for predicting the behavior of floor systems for all three types of framing analyzed. With the exception of floor systems comprised of hot-rolled primary members and secondary joist framing, the criteria presented by the Steel Construction Institute is second best for studying and predicting floor performance. The Modified Reiher-Meister scale and Murray Criterion are not very good for predicting floor acceptability for all types of framing schemes evaluated.

Recommendations. Based on findings from the criteria comparisons, it is recommended that the methods provided by the AISC Design Guide be used to evaluate the acceptability of office floor systems due to occupant-induced vibrations.

CHAPTER IV – AISC AND SCI ACCELERATIONS COMPARISON

4.1 INTRODUCTION

The final part of this study compares predicted accelerations for a number of case studies calculated using the American Institute of Steel Construction Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) and enhancements to the Steel Construction Institute criteria outlined in *Design Guide for Vibrations of Long Span Composite Floors* (Hicks, et al. 2000). The case studies are of buildings under construction or being occupied in the United States. The predicted accelerations and floor behavior acceptability are compared to assess the line of agreement between the AISC and SCI methods.

This chapter provides a comparison of predicted accelerations and acceptability from 78 case studies involving floors currently being occupied and constructed in the United States. The type of framing used during construction differentiates the structural floor systems. The classifications for framing are “hot-rolled beams”, “hot-rolled beams with joists”, “hot-rolled and castellated beams”, and “joist girders with joists”. The controlling girder, which allows the most deflection and thus the lower frequency, is considered when classifying a floor system with two different types of girders used in the framing. The bays for these case studies are modeled using the design software FLOORVIB2 (Murray and Elhouar 1994) to predict accelerations and acceptability. The FLOORVIB2 software is based on the AISC Design Guide. In addition, accelerations are also predicted using the methods outlined in the SCI Publication *Design Guide for Vibrations of Long Span Composite Floors* (Hicks, et al. 2000). The predicted accelerations for the AISC and SCI methods are then compared for agreement. This acceptability comparison is based on future or final occupancy of the case study offices. Many of the floor systems evaluated in Chapter I were under construction with very light

loading during testing. When modeling these case studies for acceleration estimates, damping is assumed to be 3 % with 4 psf of dead load and 11 psf of live load. The actual office fit-out and loading conditions acting on the floor systems were noted and used for modeling purposes for floor systems that were occupied during testing. The values for damping and loading are based on the subjective expertise of the measurement team.

Supplemental material for this report is found in the publication by Boice and Murray (2003) and contains summary sheets of the case studies used in the frequency and criteria comparisons. These sheets include floor systems properties such as damping, framing, loading, predicted and measured fundamental frequencies, accelerations, and slab properties. In addition, this supplemental material contains framing plans for the floor systems evaluated in this chapter. Appendix A provides calculations for determining floor acceptability based on the Steel Construction Institute criteria.

4.2 AISC VS. SCI ACCEPTABILITY COMPARISON

Tables 4.1 through 4.4 illustrate the results from the acceleration comparison acceptability evaluations for the assessed case studies. The peak acceleration limit of 0.50 % of gravity, shown in Figure 1.8, is used in the AISC Design Guide to rate a floor for acceptability. In this comparison, the case studies are assumed to be “General Offices” when the SCI method is used to evaluate floor acceptability. Therefore, the response factor is 8 and Equations 1.21 and 1.22 control the limiting rms acceleration. The “General Office” is “appropriate for floors where normal activities and text operations on computers are performed” (Wyatt 1989). In the predicted acceleration columns of Tables 4.1 through 4.4, the gray shading indicates that the predicted acceleration is less than or equal to the limiting acceleration and the corresponding floor system should be acceptable for occupant-induced vibrations. The gray shading in the agreement column in Tables 4.1 through 4.4 illustrates that the predictions on acceptability using the AISC Design Guide and the SCI methods concur. The agreement column uses a scale, which begins with “Strongly Disagree”, followed by “Disagree”, “Agree”, then “Strongly Agree”. The phrase “Strongly Disagree” means that the solutions between the AISC and SCI methods differ by more than 15 % of each other. The term “Disagree” means that the solutions between the AISC and SCI methods differ

within 15 % of each other. The phrase “Agree” means that the solutions between the AISC and SCI methods concur by more than 15 % each other. The phrase “Strongly Agree” means that the solutions between the AISC and SCI methods concur within 15 % of each other.

Table 4.1 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Shapes

Building	Bay	β (%)	Predicted f_{bay}		Acceleration Limits		Predicted Accelerations			Agreement
			(Hz)		D.G. 11	SCI 076	a_{peak}/g (%)		a_{rms}/g (%)	
			D.G. 11	SCI 076	a_{peak}/g (%)	a_{rms}/g (%)	D.G. 11	SCI 076	SCI 076	
The Jersey Lil Club	Typ.	3.0	3.15	3.23	0.50	0.41	0.34	0.80	0.57	Disagree
ISU	1	2.5	3.19	4.07	0.50	0.41	1.08	0.53	0.37	Strongly Disagree
	2,3	2.5	3.82	4.20	0.50	0.41	0.71	0.54	0.38	Strongly Disagree
	4	2.5	3.82	4.12	0.50	0.41	0.71	0.54	0.38	Strongly Disagree
Newgen Building	Ext.	3.0	4.64	5.60	0.50	0.41	0.52	0.55	0.39	Disagree
Penn State	Typ.	3.0	4.23	5.68	0.50	0.41	0.42	0.58	0.41	Agree
Oracle Building	1,2,3,4	2.5	3.81	4.89	0.50	0.41	0.86	0.60	0.42	Agree
	5	2.5	3.88	4.93	0.50	0.41	0.94	0.64	0.45	Agree
42 nd St. Building	Typ.	3.0	3.38	3.76	0.50	0.41	0.60	0.59	0.42	Agree
Mercy Health Ctr.	Int.	3.0	6.09	6.92	0.50	0.41	0.26	0.36	0.25	Strongly Agree
	Ext.	3.0	6.09	7.06	0.50	0.41	0.25	0.36	0.25	Strongly Agree
Newport V	Typ.	2.5	3.38	3.84	0.50	0.41	0.46	0.34	0.24	Agree
Doylestown Hosp.	Typ.	3.0	4.69	5.51	0.50	0.41	0.31	0.25	0.18	Agree
UT Med. Center	C	3.0	7.73	9.22	0.50	0.47	0.27	0.50	0.35	Agree
	D	3.0	5.66	6.27	0.50	0.41	0.23	0.40	0.28	Agree
Stanley D. Lindsey	Typ.	3.0	3.29	3.96	0.50	0.41	0.40	0.27	0.19	Agree
One Texas Court	1	2.5	3.96	4.97	0.50	0.41	0.55	0.85	0.60	Agree
	2	2.5	4.61	5.64	0.50	0.41	0.60	0.52	0.37	Disagree
AMEX	Typ.	2.5	4.24	4.24	0.50	0.41	0.56	0.71	0.50	Strongly Agree
GTE	Typ.	3.0	4.00	5.08	0.50	0.41	0.43	0.44	0.31	Strongly Agree
Three Parkwood	1	3.0	4.20	5.70	0.50	0.41	0.59	0.68	0.48	Strongly Agree
	2	3.0	3.65	4.45	0.50	0.41	0.75	0.85	0.60	Strongly Agree
Norman Pointe 1	1,4	3.0	4.35	5.72	0.50	0.41	0.38	0.45	0.32	Strongly Agree
	3	3.0	3.39	4.39	0.50	0.41	0.37	0.34	0.24	Agree
Two Parkwood	Typ.	3.0	4.58	5.79	0.50	0.41	0.57	0.72	0.51	Strongly Agree
Virginia Tech Physics	1	3.0	4.64	5.91	0.50	0.41	0.42	0.56	0.40	Strongly Agree
	2	3.0	4.60	5.81	0.50	0.41	0.42	0.55	0.39	Strongly Agree

4.2.1 FLOORS WITH HOT-ROLLED SHAPES

The accelerations and acceptability from 27 case studies with hot-rolled framing are analyzed in this subsection. From Table 4.1, 21 of 27 (78 %) of the case studies with hot-rolled framing have concurring predictions, indicating fair correlation between the AISC and SCI methods. For 10 of 27 (37 %) of the case studies constructed of hot-rolled shapes are predicted to have concurring acceptability solutions with a classification of “Strongly Agree”. For 11 of 27 (41 %) of the case studies are classified as “Agree”. However, 6 of 27 (22 %) of the case studies with the same framing scheme did not have

acceptability predictions in accord between the two methods. Of these, 3 of 27 (11 %) of these case studies are predicted to have differing acceptability solutions with a classification of “Disagree”. For 3 of 27 (11 %) of the case studies, the classification is “Strongly Disagree”.

Many of the case studies with differing vibrations behavior have higher predicted accelerations using the AISC than SCI predictions. Differences between the acceleration predictions arise from variations of assumptions on the mass participating in the vibration of the floor systems. For these specific case studies, the effective lengths and widths, L_{eff} and S , used in Equation 1.31 are larger using the SCI methods, which increases the mass and reduces the peak and rms accelerations.

The Steel Construction Institute method predicts a high peak acceleration for the Jersey Lil Building because the predicted bay frequency of 3.23 Hz is below 3.55 Hz. The only Fourier coefficient that is considered in Equation 1.27 when the fundamental frequency of a floor system is greater than or equal to 3.55 Hz is $\alpha_n = 0.10$. This Fourier coefficient is “appropriate for walking frequencies greater than the first harmonic” (Hicks, et al. 2000). The first harmonic is considered and $\alpha_n = 0.40$ when evaluating floors with predicted frequencies of less than 3.55 Hz. Therefore, the peak acceleration is increased by a factor of four for the Jersey Lil Building causing disagreement between the acceptability predictions.

4.2.2 FLOORS WITH HOT-ROLLED GIRDERS AND JOISTS

The accelerations and acceptability from 26 case studies with hot-rolled girders and joist secondary members are analyzed in this subsection. From Table 4.2, 19 of 26 (73 %) of the case studies have concurring predictions, indicating fair correlation between the AISC and SCI methods. For 11 of 26 (42 %) of the case studies constructed are predicted to have concurring acceptability solutions with a classification of “Strongly Agree”. For 8 of 26 (31 %) of the case studies are classified with “Agree”. However, 7 of 26 (27 %) of the case studies did not have acceptability predictions in accord between the two methods. Of these, one (4 %) of these case studies has differing acceptability solutions with a classification of “Disagree”. For 6 of 26 (23 %) of the case studies, the classification is “Strongly Disagree”.

Table 4.2 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Girders and Joists

Building	Bay	β (%)	Predicted f_{bay}		Acceleration Limits		Predicted Accelerations			Agreement
			(Hz)		D.G. 11	SCI 076	a_{peak}/g (%)		a_{rms}/g (%)	
			D.G. 11	SCI 076	a_{peak}/g (%)	a_{rms}/g (%)	D.G. 11	SCI 076	SCI 076	
Orange Synagogue	Typ.	2.5	5.36	6.35	0.50	0.41	0.99	2.16	1.53	Agree
Premiere Club	Typ.	2.0	5.90	6.63	0.50	0.41	0.28	0.63	0.45	Strongly Disagree
ACTIC Building	Typ.	3.0	5.55	5.55	0.50	0.41	0.29	0.57	0.40	Agree
Camron Univ.	Rm.271	3.0	5.65	6.16	0.50	0.41	0.35	0.68	0.48	Strongly Disagree
	Rm.273	3.0	5.39	5.88	0.50	0.41	0.35	0.62	0.44	Strongly Disagree
Melroe Corp. Head	Typ.	3.0	3.80	4.57	0.50	0.41	0.74	0.86	0.61	Strongly Agree
Steel Case	Mezz.	2.5	3.59	4.45	0.50	0.41	0.89	0.55	0.39	Strongly Disagree
	Int.	2.5	3.19	3.46	0.50	0.41	0.51	2.27	1.61	Agree
Patterson Ave Fac.	4	3.0	5.97	6.58	0.50	0.41	0.42	0.47	0.33	Strongly Agree
	5	3.0	5.75	6.58	0.50	0.41	0.45	0.48	0.34	Strongly Agree
	7	3.0	4.68	5.47	0.50	0.41	0.62	0.45	0.32	Disagree
UT Med. Center	A	3.0	4.53	4.53	0.50	0.41	0.34	0.48	0.34	Strongly Agree
	B	3.0	4.53	4.53	0.50	0.41	0.35	0.49	0.35	Strongly Agree
InterQuest/ Ford	Typ.	2.5	3.97	5.04	0.50	0.41	0.73	0.90	0.64	Agree
XILINX	Typ.	2.5	3.93	5.05	0.50	0.41	0.61	0.78	0.55	Strongly Agree
Civic Center	1 (ldd.)	2.5	5.16	5.16	0.50	0.41	0.73	1.28	0.91	Agree
	2 (uld.)	2.5	5.26	5.26	0.50	0.41	0.73	1.32	0.93	Agree
	3	2.5	4.70	4.70	0.50	0.41	0.66	0.82	0.58	Agree
Gateway 800 Bridge	Typ.	3.0	2.22	2.52	0.50	0.41	0.47	1.20	0.85	Strongly Disagree
MidWest Kentworth	Typ.	2.5	4.04	4.04	0.50	0.41	0.57	0.69	0.49	Strongly Agree
Ruby Tuesday HQ	1	3.0	3.99	3.99	0.50	0.41	0.34	0.39	0.28	Strongly Agree
	2	3.0	3.43	3.43	0.50	0.41	0.38	1.56	1.10	Strongly Disagree
Nations Bank	1,4,5	3.0	3.56	3.56	0.50	0.41	0.34	0.35	0.25	Strongly Agree
	2,3,6	3.0	2.95	3.10	0.50	0.41	0.42	0.53	0.37	Strongly Agree
Eight Parkwood	2,6	3.0	5.09	6.04	0.50	0.41	0.66	0.98	0.69	Agree
	3	3.0	3.49	3.92	0.50	0.41	0.47	0.58	0.41	Strongly Agree

Several factors contribute to disagreement between the AISC Design Guide and the SCI approach to floor vibrations for hot-rolled girders and joist framing. For example, like the Jersey Lil Building, the predictions of peak system accelerations for the Gateway 800 Bridge and Bay 2 of the Ruby Tuesday Headquarters are high using SCI methods. From Table 4.2, the predicted system frequencies for the Gateway 800 Bridge and Ruby Tuesday Headquarters according to the SCI method are 2.52 Hz and 3.43 Hz, respectively. The first harmonic is considered for these cases and $\alpha_n = 0.40$ is used in Equation 1.27 when evaluating floors with predicted frequencies less than 3.55 Hz. Therefore, the peak acceleration and rms acceleration is increased by a factor of four for the Gateway 800 Bridge and the Ruby Tuesday Headquarters causing disagreement between the acceptability predictions. Also, for some case studies with differing acceptability predictions, predicted peak accelerations using the SCI methods are higher

when compared to predictions based on the AISC Design Guide. These case studies include the Premier Club and the Camron University bays. The effective lengths, L_{eff} , used in Equation 1.31, limits the predicted effective mass participating in the motion of these floor systems below that used in the AISC predictions.

4.2.3 FLOORS WITH JOIST GIRDERS AND JOISTS

The accelerations and acceptability from 19 case studies with joist girders and joist framing are analyzed in this subsection. From Table 4.3, of the case studies with joist girders and joist secondary framing, 18 of 19 (95 %) have concurring predictions. For 3 of 19 (16 %) of the case studies, the acceptability solutions are classified as “Strongly Agree”. For 13 of 16 (79 %), the classification is “Agree”. Only one case study has differing acceptability predictions and a classification of “Disagree”. Based on the results of this comparison, the correlation between the AISC and SCI methods for predicted floor acceptability is very good.

Table 4.3 – AISC vs. SCI Acceptability for Framing Using Joist Girders and Joists

Building	Bay	β (%)	Predicted f_{bay}		Acceleration Limits		Predicted Accelerations			Agreement
			(Hz)		D.G. 11	SCI 076	a_{peak}/g (%)		a_{rms}/g (%)	
			D.G. 11	SCI 076	a_{peak}/g (%)	a_{rms}/g (%)	D.G. 11	SCI 076	SCI 076	
Z.C.M.I. Dept. Store	1-4,7,8,11	3.0	4.72	5.16	0.50	0.41	0.88	1.56	1.10	Agree
	9	3.0	5.00	5.51	0.50	0.41	0.81	1.55	1.10	Agree
	5,10	3.0	4.03	4.37	0.50	0.41	0.95	2.15	1.52	Agree
Patterson Ave Fac.	6	3.0	4.67	4.72	0.50	0.41	0.53	1.04	0.74	Agree
Toyota Motor Corp	Typ.	2.0	4.53	4.94	0.50	0.41	0.66	0.86	0.61	Strongly Agree
Sony Corp Building	Typ.	2.5	3.45	3.83	0.50	0.41	0.96	1.15	0.81	Agree
Diversa Building	Ext.	3.0	3.43	4.02	0.50	0.41	1.12	0.76	0.54	Agree
Pacific Tech	Typ.	3.0	3.74	4.44	0.50	0.41	0.77	0.87	0.62	Strongly Agree
Garmin Int. Bld.	Int./Ext.	2.5	3.66	3.94	0.50	0.41	1.08	1.45	1.03	Agree
IBM	Typ.	2.5	6.78	6.78	0.50	0.41	0.54	1.09	0.77	Agree
Southern Bell	2,5,7	3.0	3.91	4.51	0.50	0.41	0.48	0.70	0.49	Disagree
	9	3.0	4.54	4.77	0.50	0.41	0.54	1.04	0.74	Agree
Wholesale Grocers	1	3.0	2.45	2.45	0.50	0.41	0.21	0.57	0.40	Agree
	3	3.0	2.53	2.53	0.50	0.41	0.21	0.57	0.40	Agree
	5	3.0	2.03	2.03	0.50	0.41	0.22	0.51	0.36	Agree
Eight Parkwood	1,4,5	3.0	3.64	3.69	0.50	0.41	0.60	0.70	0.49	Strongly Agree
R.S.I. Bldg.	1,2	2.0	8.54	8.54	0.50	0.44	0.97	5.57	3.94	Agree
	3	3.0	9.61	9.61	0.50	0.49	0.68	3.08	2.18	Agree
	4	5.0	6.87	6.87	0.50	0.41	0.54	1.56	1.10	Agree

For many of the case studies constructed of joist girders and joists the AISC and SCI methods predict unacceptable floor systems. Joists and joist girders lack continuity effects between bays, and therefore, the effective length, the effective mass of these floor

systems in the direction of the secondary framing is not as large as for other floor systems. The accelerations increase with reduced effective mass participating in the predicted behavior, resulting in increased predicted peak and rms accelerations.

4.2.4 HOT-ROLLED GIRDERS AND CASTELLATED BEAMS

The accelerations and acceptability from 6 case studies with hot-rolled and castellated framing are analyzed in this subsection. From Table 4.4, of the case studies with hot-rolled girders and castellated beams, all had concurring predictions. Four of the case studies constructed of hot-rolled girders and castellated beams are predicted to have concurring acceptability solutions with a classification of “Strongly Agree”, and two are classified as “Agree”. Although the test sample of office floors with hot-rolled girders and castellated framing is limited, the correlation between the AISC and SCI methods for floor acceptability is very good.

Table 4.4 – AISC vs. SCI Acceptability for Framing Using Hot-Rolled Girders and Castellated Beams

Building	Bay	β (%)	Predicted f_{bay}		Acceleration Limits		Predicted Accelerations			Agreement
			(Hz)		D.G. 11	SCI 076	a_{peak}/g (%)		a_{rms}/g (%)	
			D.G. 11	SCI 076			a_{peak}/g (%)	a_{rms}/g (%)		
Lennox Garage	1,3	3.0	2.85	3.05	0.50	0.41	0.41	0.58	0.41	Agree
Wal-Mart	4,5	3.0	4.15	4.36	0.50	0.41	0.19	0.16	0.11	Strongly Agree
AMAT Building 34	A	3.0	5.84	6.58	0.50	0.41	0.10	0.12	0.08	Strongly Agree
	B	3.0	4.77	6.24	0.50	0.41	0.17	0.12	0.08	Strongly Agree
	C	3.0	3.57	4.21	0.50	0.41	0.43	0.19	0.13	Agree
	D	3.0	4.58	5.82	0.50	0.41	0.18	0.12	0.08	Strongly Agree

4.3 SUMMARY

Table 4.5 summarizes the results from the acceptability and acceleration AISC and SCI comparisons for the 78 case studies. The two prediction methods are in agreement for 82 % (64 of 78) of the case studies, and strongly disagree on only 12 % (9 of 78) of the case studies. The case studies with hot-rolled girders and castellated beams have the best correlations between behavior predictions, but the sample of floor systems is limited. Floors with joist girders and joists provide the next higher percentage of correlations, followed by floors constructed only of hot-rolled shapes, then floors with both hot-rolled girders and joist beams.

Table 4.5 – AISC vs. SCI Acceptability Comparison Summary

Agreement	Strongly Agree (%)	Agree (%)	Disagree (%)	Strongly Disagree (%)
Hot-Rolled Shapes	37	41	11	11
Hot-Rolled w/Joists	42	31	4	23
Hot-Rolled w/Cast.	67	33	0	0
Joist Girders w/Joists	16	79	5	0
All Systems	36	46	6	12

Note: Agreement refers to the percentage of cases studies evaluated using the AISC Design Guide 11 and the SCI method with concurring or differing acceptability predictions.

Differences between acceleration predictions arise from variations of assumptions on the mass participating in the vibration of the floor systems. For example, when using the SCI method, the effective lengths, L_{eff} , used in Equation 1.31, result in relatively small effective mass participating in the motion of floor systems with hot-rolled girders and joists. A larger participating mass is predicted by the AISC method. Another difference between the methods resides with predictions of low frequency floors. When using the SCI method, only the first harmonic is considered and $\alpha_n = 0.40$ is used in Equation 1.27, when evaluating floors with predicted frequencies less than 3.55 Hz. Therefore, the relative peak and subsequent the rms accelerations are increased by a factor of four with respect to floors having frequencies greater than 3.55 Hz.

CHAPTER V – CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

The purpose of this study was to suggest methods for improving the correlation between predicted and actual field behavior of office floor systems due to occupant-induced walking excitations. The focus of this research was divided into three main topics in which case studies were evaluated. The first focused on comparing field-measured frequencies with predicted natural frequencies using the methods outlined in the AISC Design Guide and the SCI Publication *Design Guide on the Vibration of Floors* (Wyatt 1989). The second is a comparison of current acceptability criteria and subjective field analyses of observed floor system behavior. The four acceptability criteria used to evaluate these case studies were the AISC Design Guide, enhancements to the Steel Construction Institute criteria outlined in *Design Guide for Vibrations of Long Span Composite Floors* (Hicks, et al. 2000) the Murray Criterion, and the Modified Reiher-Meister scale. Comparison of predicted accelerations and acceptability from the AISC and the SCI criteria, were assessed in the final topic.

5.2 CONCLUSIONS

5.2.1 NATURAL FREQUENCY STUDY

The purpose of this part of the study was to compare predicted and measured frequencies of floor systems for 103 case studies currently occupied and under construction in the United States and Europe. This comparison was compiled in an attempt to improve predictions of floor response due to occupant-induced excitations. Floor system properties and framing for these case studies were scrutinized using AISC and SCI methods to find correlations between predicted and measured frequencies. Based on the results of the comparisons, it can be reasonably concluded that the predicted bay frequency using Dunkerly's estimate (f_n^2) is not the most accurate method for predicting the system frequency using the AISC Design Guide procedures for all types of

framing analyzed. The predicted beam frequency using AISC methods provided sound correlations with the measured bay frequencies, although the scatter of the data was modest. On the other hand, with the exception of floor systems with joist girders and joists, the results showed that the SCI methods provided more accurate predictions of bay frequency despite a fair amount of data scatter. In addition, revisions to Dunkerly's estimate, f_n^4 (Equation 2.1) and f_n^6 (Equation 2.2), marginally improved the correlation for predicting system frequencies using between predicted and measured frequencies for both the AISC Design Guide and SCI methods. However, the scatter of data for these modified system predictions were moderately high.

5.2.2 FLOOR VIBRATION CRITERIA COMPARISON

The primary goal of this part of the research was to determine which floor vibration criteria best predicts floor response. From the results of the four methods used to examine the case studies, the AISC Design Guide criterion is judged the most accurate method for predicting floor vibrations acceptability. The SCI criterion is the next most consistent method, followed by the Murray Criterion then the Modified Reiher-Meister scale.

5.2.3 AISC AND SCI ACCELERATION COMPARISONS

The final part of this study compares predicted accelerations and floor behavior tolerability for 78 case studies evaluated using the AISC and the SCI criteria. The two prediction methods are in agreement for 82 % (64 of 78) of the case studies, and strongly disagree for only 12 % (9 of 78) of the case studies. The case studies with hot-rolled girders and castellated beams have the best correlations between behavior predictions, but the sample of floor systems is very limited. Floors with joist girders and joists provide the next higher percentage of correlations, followed by floors constructed only of hot-rolled shapes, then floors with hot-rolled girders and joist beams.

5.3 COMMENTS

When evaluating a floor system for acceptability using the SCI criteria, several considerations should be made for the effective length, L_{eff} , used in the effective mass, M , Equation 1.31. Floor systems comprised solely of hot-rolled framing members should

strictly use the guidelines presented in Table 1.1 for determining effective lengths and widths. However, when floors are comprised of hot-rolled shapes with joists or joist girders with joists, continuity effects of secondary framing members need to be considered. Table 1.1 should be used as outlined by SCI when the predicted motion of the secondary beams controls the floor system behavior. When the predicted motion of the floor system is controlled by the primary framing and the relative flexibility, RF_{main_beam} , is greater than 0.60, only the length of the secondary beam should be used as L_{max} and subsequently L_{eff} . The term L_{max} is defined as the total length of the secondary beam when considered to act continuously. Unlike hot-rolled secondary framing, joists are not considered to act continuously from bay to bay. Another recommendation that should be considered when using the SCI criteria for floor acceptability is that actual corridor dimensions for computing the resonant build-up factor, R_I , should be used. The resonant build-up factor, Equation 1.28, is a function of D . The property D is taken conservatively as the longer of the floors plan dimensions or, when known, the longest architectural corridor length. The architectural corridor lengths were not known and the resonant build-up factors were calculated to be 1.0 for the case studies evaluated. The resonant build-up factor is used as a reduction of the peak acceleration. In future studies, architectural corridor features should be noted to provide more accurate predictions of floor behavior using SCI methods.

5.4 AREAS FOR FURTHER RESEARCH

Many of the floor systems in the natural frequency study showed the best correlation between predicted beam frequency and measured bay frequency when evaluated using the AISC Design Guide. Future floor vibration research should study the framing member properties that would cause the girder frequency to be neglected when calculating bay frequencies for each type of framing scheme. These member properties should include span-to-depth ratios for girders and beams, as well as, girder span-to-beam span ratios. Moreover, this study showed that the modifications to Dunkerly's equation, f_n^4 and f_n^6 , on average marginally provide more reliable predictions than estimations using f_n^2 for AISC and SCI methods. Further study of these modifications is recommended for both methods. In addition, only six case studies of floors with hot-rolled girders and

castellated beams were used in this study. Therefore, in future research, a broader sample of floor systems with hot-rolled girders and castellated beams should be considered to compare measured and predicted frequencies, accelerations, and behavioral acceptability.

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APPENDIX A – EXAMPLE CALCULATIONS

APPENDIX A.1 AISC DESIGN GUIDE 11 CALCULATIONS

Appendix A.1 AISC Design Guide 11 Calculations

A.1.1 HOT-ROLLED SHAPES

Newport V
File # 596
Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 01/22/03 By: MDB
Job Id: Newport Office Center V
Id: J-K/J-4

Page 1

Span= 30.833 ft Left Girder: Wall	Beam W21x44 3 Spc @ 123.33	() Mezzanine () Beam/OS () Girder/OS Right Girder: W27x84	Defaults Table: FLOORVIB.VDT Floor Width: 200 ft Floor Length: 129.33 ft () Beam Continuity () Truss or Joist Girder Continuity Acc. Limit: 0.5 % Modal Damping: 0.025
Left Bm. Span: 0.0 ft	Bm Span= 45.83 ft	Right Bm. Span: 37.667 ft	
Concrete: dc: 5-1/4 in. (total depth) fc': 3.5 Ksi wt.: 115 pcf	Deck Height: 2" Loading: Dead: 4 psf Collateral: 12.5 psf (V) Live: 8 psf	Activity: WALKING Occupancy: OFFICE OR RESIDENCE Design Criteria: AISC Guide, Chapter 4 System Evaluation: SATISFIES CRITERION Combined Mode $a_p/g=0.46\%$ $\leq 0.50\%$	

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Office or Residence
Evaluation Criterion: Walking, AISC Design Guide, Chapter 4
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due To Human Activity",
Design Guide, June 1997

Constant Force, $P_o = 65$ lb
Modal Damping Ratio, $\xi = 0.025$
Acceleration Limit, $a_o/g \times 100\% = 0.50\%$
Joist bottom chords are not extended
Girders are not continuous at columns

Section	w, plf	Itr, in4	f, Hz.
Beam W21x44	724.4	2825.3	3.78
Left Wall			
Right Girder W27x84	3027.0	7841.1	6.80
Bay (Using smaller girder frequency)			3.38

Evaluation:

-- Continued --

Appendix A.1.1

AISC Design Guide 11 Calculations

Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/22/03 By: MDB
Job Id: Newport Office Center V
Id: J-K/J-4

Page 2

Combined Mode $\Delta_p/g = 0.46 \% \leq 0.50 \%$

The system SATISFIES THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 41.7 psf
Dead loads = 4.0 psf
Collateral loads = 12.5 psf
Live loads = 8.0 psf
Actual beam and girder weights

Tributary width for left girder = $45.83/2 = 22.92$ ft.
Tributary width for right girder = $45.83/2 + 37.67/2 = 41.75$ ft.

CONCRETE/SLAB DATA:

Concrete $d_c = 5.25$ in. $f_c' = 3.5$ Ksi
 $w_t = 115$ pcf $E_c = 2307$ Ksi

Modular ratio, $n = E_s / (1.35 E_c) = 9.31$

Deck height: 2 in.
Effective concrete thickness in deck: 1 in.

BEAM CALCULATIONS:

Beam section: W21x44 $d = 20.660$ in. $A = 13.00$ in.² $I_x = 843$ in.⁴

Spacing: $S = 123.33$ in.
Span: $L_b = 45.83$ ft.
Uniform load: $w_b = (41.7 + 4.0 + 12.5 + 8.0) \times 123.33/12 + 44.0$
 $= 724.4$ plf

Transformed moment of inertia:
Effective concrete width = $\min(0.4 L_b, S) = 123.330$ in.
Effective concrete depth = 3.250 in.
Transformed concrete width = 13.245 in.
Transformed concrete area = 43.046 in.²
Distance to neutral axis = 10.718 in. (Above beam c. g.)
Transformed moment of inertia = 2825.3 in.⁴

$$\delta_b = \frac{5 w_b L_b^4}{384 E_s I_b} = \frac{5 \times 724.4 \times 45.83^4 \times 1728}{384 \times E_s \times 2825.3} = 0.878 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_b} \right]^{0.5}$$

-- Continued --

Appendix A.1.1

AISC Design Guide 11 Calculations

Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/22/03 By: MDB
Job Id: Newport Office Center V
Id: J-K/J-4

Page 3

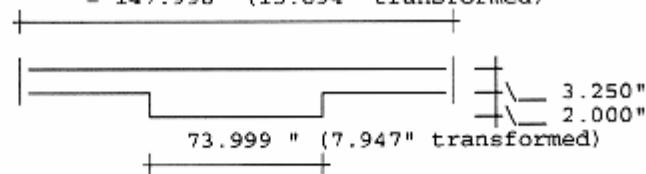
$$= 0.18 \times \left[\frac{386}{0.878} \right]^{0.5} = 3.78 \text{ Hz.}$$

Cj = 2.0
Floor Width= 200.00 ft.
Ds = (12 de^3)/(12 n) = (12 x 4.25^3)/(12 x 9.31) = 8.24 in^4/ft.
Db = Itr/S = 2825.3/10.28 = 274.90 in^4/ft.
Bb = min[Cj (Ds/Db)^0.25 Lb= 38.14 ft.; 2/3 x 200.00 ft.= 133.33 ft.]
= 38.14 ft.
Continuity Factor=1.5 since Continuous
and max. adj. Lb=37.67 ft > 0.7 Lb=32.08 ft.
Wb = 1.5 x (0.724/10.28) x 38.14 x 45.83 = 184.8 Kips

RIGHT GIRDER CALCULATIONS:

Girder section: W27x84 d= 26.710 in. A= 24.80 in.2 Ix= 2850 in.4
Tributary width = 41.75 ft.
Span: Lg = 30.83 ft.
Equivalent uniform load: wg = 41.75 x (724.4/10.28) + 84.0
= 3027.0 plf

Transformed moment of inertia:
min(0.2 Lg, 45.83x12/2) + min(0.2 Lg, 37.67x12/2)
= 147.998" (15.894" transformed)



Effective concrete width = 147.998 in. and 73.999 in.
Transformed concrete width = 15.894 in. and 7.947 in.
Transformed concrete area = 51.656 in.2 and 15.894 in.2
Distance to neutral axis = 11.968 in. (Above girder c. g.)
Transformed moment of inertia = 7841.1 in.4

$$\delta_g = \frac{5 \text{ wg Lg}^4}{384 \text{ Es Ig}} = \frac{5 \times 3027. \times 30.83^4 \times 1728}{384 \times \text{Es} \times 7841.12} = 0.271 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_g} \right]^{0.5}$$

$$= 0.18 \times \left[\frac{386}{0.271} \right]^{0.5} = 6.80 \text{ Hz.}$$

-- Continued --

Appendix A.1.1 AISC Design Guide 11 Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/22/03 By: MDB
Job Id: Newport Office Center V
Id: J-K/J-4

Page 4

$$L = 0.271 \text{ } ^\circ$$

Cg = 1.8
Floor Length = 129.33 ft.
Db = Itr/S = 2825.3/10.28 = 274.90 in⁴/ft.
Dg = Itr/Avg. Lb = 7841.1/41.75 = 187.82 in⁴/ft.
Bg = min[Cg (Db/Dg)^{0.25} Lg = 61.04 ft.; 2/3 x 129.33 ft. = 86.22 ft.]
= 61.04 ft.
Continuity Factor = 1.0 since Not Continuous
Wg = 1.0 x (3.027/41.75) x 61.04 x 30.83 = 136.5 Kips

COMBINED MODE CALCULATIONS:

$$\delta b = 0.878 \text{ in.} \quad \delta r g = 0.271 \text{ in. (Right girder controls)}$$

Because the girder span, Lg=30.83 ft, is less than Bb=38.14 ft.,
the girder deflection is reduced by the factor = Lg/Bb ≥ 0.5.

$$\max(Lg/Bb, 0.5) = \max(30.83/38.14, 0.5) = 0.808$$

Therefore:

$$\delta g = 0.808 \times 0.271 = 0.219 \text{ in.}$$

$$\text{System frequency, } f_n = 0.18 \sqrt{\frac{386}{\delta b + \delta g}} = 3.38 \text{ Hz}$$

$$W_b = 184.8 \text{ Kips} \quad W_g = 136.5 \text{ kips}$$

$$W_c = \frac{\delta b}{\delta b + \delta g} W_b + \frac{\delta g}{\delta b + \delta g} W_g$$

$$= \frac{0.878}{1.096} \times 184.8 + \frac{0.219}{1.096} \times 136.5 = 175.2 \text{ Kips} = 175167 \text{ lbs}$$

$$\xi = \text{modal damping ratio} = 0.025$$

$$(a_p/g) = P_o \exp(-0.35 f_n) / (\xi W_c)$$

$$= 65 \exp(-0.35 \times 3.38) / (0.025 \times 175167)$$

$$= 0.46 \% \leq 0.50 \% \text{ - SATISFIES CRITERION}$$

Since $f_n = 3.38 \text{ Hz} \leq 9 \text{ Hz}$ - Stiffness criterion does not need to be checked.

-- End --

Appendix A.1 AISC Design Guide 11 Calculations

A.1.2 HOT-ROLLED GIRDERS WITH JOISTS

Melroe Corporation

File # 563

Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 01/23/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 1

Span= 30 ft				() Mezzanine	Defaults
Left Girder: W24x76				() Beam/OS	Table: FLOORVIB.VDT
				() Girder/OS	Floor Width: 210 ft
				Right Girder: W24x76	Floor Length: 118.6 ft
					() Ext. Jt. Bot. Chords
					() Truss or Joist
Left Bm. Span: 40.125 ft				Right Bm. Span: 40.125 ft	Girder Continuity
					Acc. Limit: 0.5 %
					Modal Damping: 0.03
Concrete: dc: 2-3/4 in. (total depth)				Activity: WALKING	
fc: 3 Ksi				Occupancy: OFFICE OR RESIDENCE	
wt.: 145 pcf				Design Criteria: AISC Guide, Chapter 4	
Deck Height: 5/8"				System Evaluation:	
Loading: Dead: 7 psf				DON'T SATISFY CRITERION	
Collateral: psf				Combined Mode ap/g=0.74 % > 0.50 %	
(✓) Live: 11 psf				<Press [F1] for Advice>	

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Office or Residence
Evaluation Criterion: Walking, AISC Design Guide, Chapter 4
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due To Human Activity",
Design Guide, June 1997

Constant Force, $P_o = 65 \text{ lb}$
Modal Damping Ratio, $\xi = 0.03$
Acceleration Limit, $a_o/g \times 100\% = 0.50 \%$
Joist bottom chords are not extended
Girders are not continuous at columns

	Section	w, plf	Itr, in4	f, Hz.
Joist	24K8	108.5	435.1	5.46
Left Girder	W24x76	2204.4	3089.6	5.28
Right Girder	W24x76	2204.4	3089.6	5.28
Bay (Using smaller girder frequency)				3.80

Evaluation:

-- Continued --

Appendix A.1.2 AISC Design Guide 11 Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation File # 563 Typical Bay

Date: 01/23/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 2

Combined mode $ap/g = 0.74 \% > 0.50 \%$

The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 30.5 psf
Dead loads = 7.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for girder = $40.13/2 + 38.38/2 = 39.25$ ft.

CONCRETE/SLAB DATA:

Concrete dc= 2.75 in. fc'= 3 Ksi
wt= 145 pcf Ec= 3024 Ksi

Modular ratio, $n = E_s / (1.35 E_c) = 7.10$

Deck height: 0.625 in.
Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 24K8 (11.5 plf)
d= 24.000 in. A= 2.157 in.² Ix= 282 in.⁴ yc= 10.72 in.

Spacing: S = 24 in.
Span: Lj = 38.38 ft.
Uniform load: wj = $(30.5 + 7.0 + 0.0 + 11.0) \times 24.00/12 + 11.5$
= 108.5 plf

Transformed moment of inertia:

Effective concrete width = $\min(0.4 L_b, S) = 24.000$ in.
Effective concrete depth = 2.125 in.
Transformed concrete width = 3.379 in.
Transformed concrete area = 7.179 in.²
Distance to neutral axis = 9.543 in. (Above beam c. g.)
Transformed moment of inertia = 539.8 in.⁴
 $6 \leq L_j/D = 19.19 \leq 24$
 $Cr = 0.9 [1 - e^{(-0.28 L_j/D)}]^{2.8} = 0.89$
 $\tau = (1/Cr) - 1 = 0.126$
Effective moment of inertia = $1/[\tau/I_{chords} + 1/I_{comp}] = 435.1$ in.⁴

$$\delta_j = \frac{5 w_j L_j^4}{384 E_s I_j} = \frac{5 \times 108.5 \times 38.38^4 \times 1728}{384 \times E_s \times 435.1} = 0.420 \text{ in.}$$

-- Continued --

Appendix A.1.2 AISC Design Guide 11 Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/23/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 3

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_j} \right]^{0.5}$$

$$= 0.18 \times \left[\frac{386}{0.420} \right]^{0.5} = 5.46 \text{ Hz.}$$

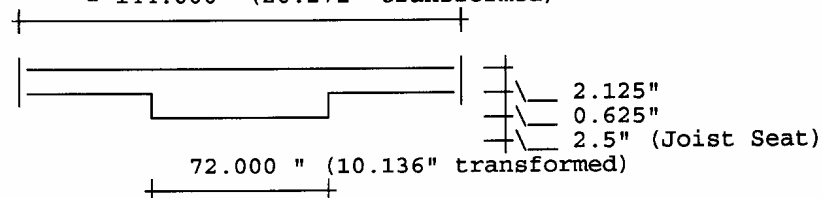
Cj = 2.0
Floor Width= 210.00 ft.
Ds = (12 de³)/(12 n) = (12 x 2.4375³)/(12 x 7.10) = 2.04 in⁴/ft.
Dj = Itr/S = 435.1/2 = 217.55 in⁴/ft.
Bj = min[Cj (Ds/Dj)^{0.25} Lj= 23.88 ft.; 2/3 x 210.00 ft.= 140.00 ft.]
= 23.88 ft.
Continuity Factor=1.0 since joist bottom chords are not extended
Wj = 1.0 x (0.108/2.00) x 23.88 x 38.38 = 49.7 Kips

LEFT GIRDER CALCULATIONS:

Girder section: W24x76 d= 23.920 in. A= 22.40 in.² Ix= 2100 in.⁴

Tributary width = 39.25 ft.
Span: Lg = 30.00 ft.
Equivalent uniform load: wg = 39.25 x (108.5/2.00) + 76.0
= 2204.4 plf

Transformed moment of inertia:
min(0.2 Lg, 40.13x12/2) + min(0.2 Lg, 38.38x12/2)
= 144.000" (20.271" transformed)



Effective concrete width = 144.000 in. and 72.000 in.
Transformed concrete width = 20.271 in. and 10.136 in.
Transformed concrete area = 43.076 in.² and 6.335 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 10.989 in. (Above girder c. g.)
Transformed moment of inertia = 6058.3 in.⁴
Effective moment of inertia = Ig+(Itr-Ig)/4 = 3089.6 in.⁴

$$\delta_g = \frac{5 w_g L_g^4}{384 E_s I_g} = \frac{5 \times 2204. \times 30.00^4 \times 1728}{384 \times E_s \times 3089.59} = 0.448 \text{ in.}$$

-- Continued --

Appendix A.1.2

AISC Design Guide 11 Calculations

Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/23/03 By: MDB
 Job Id: Melroe Corp. Building --- West Fargo, N.D.
 Id: Typ.

Page 4

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta g} \right]^{0.5}$$

$$= 0.18 \times \left[\frac{386}{0.448} \right]^{0.5} = 5.28 \text{ Hz.}$$

Cg = 1.6
 Floor Length = 118.60 ft.
 Dj = Itr/S = 435.1/2 = 217.55 in⁴/ft.
 Dg = Itr/Avg. Lj = 3089.6/39.25 = 78.72 in⁴/ft.
 Bg = min[Cg (Dj/Dg)^{0.25} Lg = 61.89 ft.; 2/3 x 118.60 ft. = 79.07 ft.]
 = 61.89 ft.
 Continuity Factor = 1.0 since Not Continuous
 Wg = 1.0 x (2.204/39.25) x 61.89 x 30.00 = 104.3 Kips

RIGHT GIRDER CALCULATIONS:

Girder section: W24x76 d = 23.920 in. A = 22.40 in.² Ix = 2100 in.⁴
 Same as Left girder.

COMBINED MODE CALCULATIONS:

Using girder with smaller frequency:

$$\delta j = 0.420 \text{ in.} \qquad \delta g = 0.448 \text{ in.}$$

$$\text{System frequency, } f_n = 0.18 \sqrt{\frac{386}{\delta j + \delta g}} = 3.80 \text{ Hz}$$

$$W_j = 49.7 \text{ Kips} \qquad W_g = 104.3 \text{ kips}$$

$$W_c = \frac{\delta j}{\delta j + \delta g} W_b + \frac{\delta g}{\delta j + \delta g} W_g$$

$$= \frac{0.420}{0.868} \times 49.7 + \frac{0.448}{0.868} \times 104.3 = 77.9 \text{ Kips} = 77900 \text{ lbs}$$

$$\beta = \text{modal damping ratio} = 0.03$$

$$(a_p/g) = P_o \exp(-0.35 f_n) / (\beta W_c)$$

$$= 65 \exp(-0.35 \times 3.80) / (0.03 \times 77900)$$

$$= 0.74 \% > 0.50 \% \text{ - DOES NOT SATISFY CRITERION}$$

-- Continued --

Appendix A.1.2
AISC Design Guide 11 Calculations
Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/23/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 5

Since $f_n = 3.80 \text{ Hz} \leq 9 \text{ Hz}$ - Stiffness criterion does not need to be checked.

Warning:

The effective girder panel width cannot exceed $2/3$ times the total floor length. Thus, the total floor length is a required input.

The length of the floor should be at least equal to the specified joist span length plus the adjoining span lengths.

Specified floor length = $118' - 7.2" < 118' - 7.5"$

(The user has elected to ignore this condition.)

Warning:

The floor system does not satisfy the specified criterion.

-- End --

Appendix A.1 AISC Design Guide 11 Calculations

A.1.3 JOIST GIRDERS WITH JOISTS

Garmin International

File # 506

Typical Exterior Bay

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 1

Span= 35 ft		() Mezzanine	Defaults
Left Girder:		() Beam/OS	Table: FLOORVIB.VDT
Wall		() Girder/OS	Floor Width: 345 ft
		Right Girder:	Floor Length: 180 ft
		36G14N4.05	
Left		Right	() Ext. Jt. Bot. Chords
Bm. Span:		Bm. Span:	() Truss or Joist
0.0 ft	Bm Span= 28 ft	28 ft	Girder Continuity
			Acc. Limit: 0.5 %
			Modal Damping: 0.025
Concrete: dc: 2-1/2 in. (total depth)		Activity: WALKING	
fc': 3.5 Ksi		Occupancy: OFFICE OR RESIDENCE	
wt.: 150 pcf		Design Criteria: AISC Guide, Chapter 4	
Deck Height: 5/8"		System Evaluation:	
Loading: Dead: 4 psf		DOES NOT SATISFY CRITERION	
Collateral: psf		Combined Mode $a_p/q=1.08\%$ > 0.50 %	
(✓) Live: 11 psf		<Press [F1] for Advice>	

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Office or Residence
Evaluation Criterion: Walking, AISC Design Guide, Chapter 4
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E,
"Floor Vibrations Due To Human Activity",
Design Guide, June 1997

Constant Force, $P_o = 65$ lb
Modal Damping Ratio, $\xi = 0.025$
Acceleration Limit, $a_o/g \times 100\% = 0.50\%$
Joist bottom chords are not extended
Girders are not continuous at columns

	Section	w, plf	Itr, in4	f, Hz.
Joist	18K4	115.4	171.8	6.25
Left Wall				
Right Joist Girder	36G14N4.05	1320.8	2508.0	4.52
Bay (Using smaller girder frequency)				3.66

Evaluation:

-- Continued --

Appendix A.1.3 AISC Design Guide 11 Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 2

Combined mode $a_p/g = 1.08 \% > 0.50 \%$

The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 28.3 psf
Dead loads = 4.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for left girder = $28.00/2 = 14.00$ ft.
Tributary width for right girder = $2 \times (28.00/2) = 28.00$ ft.

CONCRETE/SLAB DATA:

Concrete $d_c = 2.50$ in. $f_c' = 3.5$ Ksi
 $w_t = 150$ pcf $E_c = 3437$ Ksi

Modular ratio, $n = E_s / (1.35 E_c) = 6.25$

Deck height: 0.625 in.
Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 18K4 (7.2 plf)
 $d = 18.000$ in. $A = 1.431$ in.² $I_x = 102$ in.⁴ $y_c = 8.06$ in.

Spacing: $S = 30$ in.
Span: $L_j = 28.00$ ft.
Uniform load: $w_j = (28.3 + 4.0 + 0.0 + 11.0) \times 30.00/12 + 7.2$
 $= 115.4$ plf

Transformed moment of inertia:

Effective concrete width = $\min(0.4 L_b, S) = 30.000$ in.
Effective concrete depth = 1.875 in.
Transformed concrete width = 4.800 in.
Transformed concrete area = 9.000 in.²
Distance to neutral axis = 8.299 in. (Above beam c. g.)
Transformed moment of inertia = 219.0 in.⁴
 $6 \leq L_j/D = 18.67 \leq 24$
 $Cr = 0.9 [1 - e^{(-0.28 L_j/D)}]^{2.8} = 0.89$
 $r = (1/Cr) - 1 = 0.128$
Effective moment of inertia = $1 / [r / I_{chords} + 1 / I_{comp}] = 171.8$ in.⁴

$$\delta_j = \frac{5 w_j L_j^4}{384 E_s I_j} = \frac{5 \times 115.4 \times 28.00^4 \times 1728}{384 \times E_s \times 171.8} = 0.320 \text{ in.}$$

-- Continued --

Appendix A.1.3 AISC Design Guide 11 Calculations Joist Girders with Joists (Cont.)

Garmin International File # 506 Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 3

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_j} \right]^{0.5}$$

$$= 0.18 \times \left[\frac{386}{0.320} \right]^{0.5} = 6.25 \text{ Hz.}$$

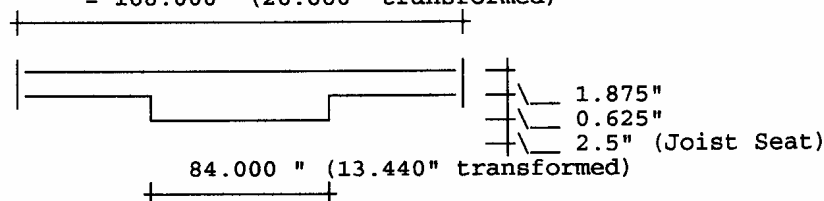
Cj = 2.0
Floor Width = 345.00 ft.
Ds = (12 de³) / (12 n) = (12 x 2.1875³) / (12 x 6.25) = 1.67 in⁴/ft.
Dj = Itr/S = 171.8/2.5 = 68.72 in⁴/ft.
Bj = min[Cj (Ds/Dj)^{0.25} Lj = 22.13 ft.; 2/3 x 345.00 ft. = 230.00 ft.]
= 22.13 ft.
Continuity Factor = 1.0 since joist bottom chords are not extended
Wj = 1.0 x (0.115/2.50) x 22.13 x 28.00 = 28.6 Kips

RIGHT GIRDER CALCULATIONS:

Joist Girder: 36G14N4.05 (27.1 plf)
A = 6.38 in.² Ix = 1929 in.⁴ yc = 16.06 in.

Tributary width = 28.00 ft.
Span: Lg = 35.00 ft.
Equivalent uniform load: wg = 28.00 x (115.4/2.50) + 27.1
= 1320.8 plf

Transformed moment of inertia:
min(0.2 Lg, 28.00x12/2) + min(0.2 Lg, 28.00x12/2)
= 168.000" (26.880" transformed)



Effective concrete width = 168.000 in. and 84.000 in.
Transformed concrete width = 26.880 in. and 13.440 in.
Transformed concrete area = 50.399 in.² and 8.400 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 17.988 in. (Above girder c. g.)
Transformed moment of inertia = 4243.7 in.⁴
Effective moment of inertia = Ig + (Itr - Ig) / 4 = 2508.0 in.⁴

$$\delta_g = \frac{5 \text{ wg } L_g^4}{48 E I_g} = \frac{5 \times 1320. \times 35.00^4 \times 1728}{48 \times 2508.0} = 0.613 \text{ in.}$$

-- Continued --

Appendix A.1.3 AISC Design Guide 11 Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 4

$$384 E_s I_g \quad 384 \times E_s \times 2507.96$$

$$\begin{aligned} \text{Frequency} &= 0.18 \times \left[\frac{g}{\delta_g} \right]^{0.5} \\ &= 0.18 \times \left[\frac{386}{0.613} \right]^{0.5} = 4.52 \text{ Hz.} \end{aligned}$$

$$C_g = 1.6$$

$$\text{Floor Length} = 180.00 \text{ ft.}$$

$$D_j = I_{tr}/S = 171.8/2.5 = 68.72 \text{ in}^4/\text{ft.}$$

$$D_g = I_{tr}/\text{Avg. } L_j = 2508.0/28.00 = 89.57 \text{ in}^4/\text{ft.}$$

$$B_g = \min[C_g (D_j/D_g)^{0.25} L_g = 52.41 \text{ ft.}; 2/3 \times 180.00 \text{ ft.} = 120.00 \text{ ft.}] = 52.41 \text{ ft.}$$

$$\text{Continuity Factor} = 1.0 \text{ since Not Continuous}$$

$$W_g = 1.0 \times (1.321/28.00) \times 52.41 \times 35.00 = 86.5 \text{ Kips}$$

COMBINED MODE CALCULATIONS:

$$\delta_j = 0.320 \text{ in.}$$

$$\delta_{rg} = 0.613 \text{ in. (Right girder controls)}$$

$$\text{System frequency, } f_n = 0.18 \sqrt{\frac{386}{\delta_j + \delta_g}} = 3.66 \text{ Hz}$$

$$W_j = 28.6 \text{ Kips}$$

$$W_g = 86.5 \text{ kips}$$

$$W_c = \frac{\delta_j}{\delta_j + \delta_g} W_b + \frac{\delta_g}{\delta_j + \delta_g} W_g$$

$$= \frac{0.320}{0.934} \times 28.6 + \frac{0.613}{0.934} \times 86.5 = 66.6 \text{ Kips} = 66648 \text{ lbs}$$

$$\xi = \text{modal damping ratio} = 0.025$$

$$(a_p/g) = P_o \exp(-0.35 f_n) / (\xi W_c)$$

$$= 65 \exp(-0.35 \times 3.66) / (0.025 \times 66648)$$

$$= 1.08 \% > 0.50 \% \text{ - DOES NOT SATISFY CRITERION}$$

Since $f_n = 3.66 \text{ Hz} \leq 9 \text{ Hz}$ - Stiffness criterion does not need to be checked.

Warning:

The floor system does not satisfy the specified criterion.

-- End --

Appendix A.1 AISC Design Guide 11 Calculations

A.1.4 HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS

AMAT Building 34

File # Boice7

Bay "B"

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 01/22/03 By: MDB
Job Id: AMAT Building 34 --- Austin, TX
Id: "B" as Tested

Page 1

Span= 40 ft	() Mezzanine	Defaults
Left Girder: W36x182	() Beam//OS	Table: FLOORVIB.VDT
	() Girder//OS	Floor Width: 120 ft
	Right Girder: W36x182	Floor Length: 120 ft
		() Beam Continuity
Left Bm. Span: 40 ft	Right Bm. Span: 40 ft	() Truss or Joist
Bm Span= 40 ft		Girder Continuity
		Acc. Limit: 0.5 %
		Modal Damping: 0.01
Concrete: dc: 7 in. (total depth)	Activity: WALKING	
fc: 4 Ksi	Occupancy: OFFICE OR RESIDENCE	
wt.: 145 pcf	Design Criteria: AISC Guide, Chapter 4	
Deck Height: 2"	System Evaluation:	
Loading: Dead: psf	DOES NOT SATISFY CRITERION	
Collateral: psf	Combined Mode $a_p/g=0.53\%$ > 0.50 %	
(V) Live: 0.001 psf	<Press [F1] for Advice>	

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Office or Residence
Evaluation Criterion: Walking, AISC Design Guide, Chapter 4
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E,
"Floor Vibrations Due To Human Activity",
Design Guide, June 1997

Constant Force, $P_o = 65$ lb
Modal Damping Ratio, $\delta = 0.01$
Acceleration Limit, $a_o/g \times 100\% = 0.50\%$
Joist bottom chords are not extended
Girders are not continuous at columns

	Section	w, plf	I _{tr} , in ⁴	f, Hz.
Beam	CB27x35/46	617.8	4717.0	6.93
Left Girder	W36x182	3270.9	32273.6	7.88
Right Girder	W36x182	3270.9	32273.6	7.88
Bay (Using smaller girder frequency)				5.21

Evaluation:

-- Continued --

Appendix A.1.4

AISC Design Guide 11 Calculations

Hot-Rolled Girders with Castellated Beams (Cont.)

AMAT Building 34 File # Boice7 Bay "B"

Date: 01/22/03 By: MDB
Job Id: AMAT Building 34 --- Austin, TX
Id: "B" as Tested

Page 2

Combined mode $ap/g = 0.53 \% > 0.50 \%$

The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 73.5 psf
Dead loads = 0.0 psf
Collateral loads = 0.0 psf
Live loads = 0.001 psf
Actual beam and girder weights

Tributary width for girder = $2 \times (40.00/2) = 40.00 \text{ ft.}$

CONCRETE/SLAB DATA:

Concrete dc= 7.00 in. fc'= 4 Ksi
wt= 145 pcf Ec= 3492 Ksi

Modular ratio, $n = E_s / (1.35 E_c) = 6.15$

Deck height: 2 in.
Effective concrete thickness in deck: 1 in.

SUPPORTED MEMBER CALCULATIONS:

Built-up member: CB27x35/46 (29.8 plf)
A= 8.750 in.² Ix= 1407 in.⁴ yc= 15.50 in.

Spacing: S = 96 in.
Span: Lb = 40.00 ft.
Uniform load: wb = $(73.5 + 0.0 + 0.0 + 0.001) \times 96.00/12 + 29.8$
= 617.8 plf

Transformed moment of inertia:
Effective concrete width = $\min(0.4 L_b, S) = 96.000 \text{ in.}$
Effective concrete depth = 5.000 in.
Transformed concrete width = 15.606 in.
Transformed concrete area = 78.028 in.²
Distance to neutral axis = 17.985 in. (Above beam c. g.)
Transformed moment of inertia = 4717.0 in.⁴

$$\delta_b = \frac{5 w_b L_b^4}{384 E_s I_b} = \frac{5 \times 617.8 \times 40.00^4 \times 1728}{384 \times E_s \times 4717.0} = 0.260 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_b} \right]^{0.5}$$

-- Continued --

Appendix A.1.4 AISC Design Guide 11 Calculations Hot-Rolled Girders with Castellated Beams (Cont.)

AMAT Building 34 File # Boice7 Bay "B"

Date: 01/22/03 By: MDB
Job Id: AMAT Building 34 --- Austin, TX
Id: "B" as Tested

Page 3

$$= 0.18 \times \left[\frac{386}{0.260} \right]^{0.5} = 6.93 \text{ Hz.}$$

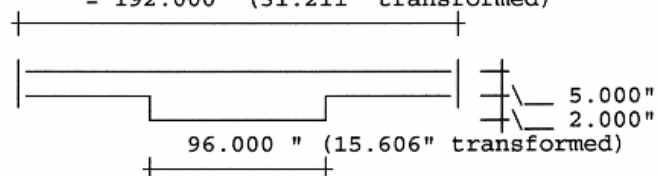
Cj = 2.0
Floor Width = 120.00 ft.
Ds = (12 de^3)/(12 n) = (12 x 6^3)/(12 x 6.15) = 35.11 in^4/ft.
Db = Itr/S = 4717.0/8.00 = 589.62 in^4/ft.
Bb = min[Cj (Ds/Db)^0.25 Lb = 39.52 ft.; 2/3 x 120.00 ft. = 80.00 ft.]
= 39.52 ft.
Continuity Factor = 1.5 since Continuous
and max. adj. Lb = 40.00 ft > 0.7 Lb = 28.00 ft.
Wb = 1.5 x (0.618/8.00) x 39.52 x 40.00 = 183.1 Kips

LEFT GIRDER CALCULATIONS:

Girder section: W36x182 d = 36.330 in. A = 53.60 in^2 Ix = 11300 in^4

Tributary width = 40.00 ft.
Span: Lg = 40.00 ft.
Equivalent uniform load: wg = 40.00 x (617.8/8.00) + 182.0
= 3270.9 plf

Transformed moment of inertia:
min(0.2 Lg, 40.00x12/2) + min(0.2 Lg, 40.00x12/2)
= 192.000" (31.211" transformed)



Effective concrete width = 192.000 in. and 96.000 in.
Transformed concrete width = 31.211 in. and 15.606 in.
Transformed concrete area = 156.056 in^2 and 31.211 in^2
Distance to neutral axis = 17.168 in. (Above girder c. g.)
Transformed moment of inertia = 32273.6 in^4

$$\delta g = \frac{5 \text{ wg Lg}^4}{384 \text{ Es Ig}} = \frac{5 \times 3270. \times 40.00^4 \times 1728}{384 \times \text{Es} \times 32273.62} = 0.201 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta g} \right]^{0.5}$$

$$= 0.18 \times \left[\frac{386}{0.260} \right]^{0.5} = 7.88 \text{ Hz.}$$

-- Continued --

Appendix A.1.4

AISC Design Guide 11 Calculations

Hot-Rolled Girders with Castellated Beams (Cont.)

AMAT Building 34

File # Boice7

Bay "B"

Date: 01/22/03 By: MDB
 Job Id: AMAT Building 34 --- Austin, TX
 Id: "B" as Tested

Page 4

L 0.201 J

Cg = 1.8
 Floor Length= 120.00 ft.
 Db = Itr/S = 4717.0/8.00 = 589.62 in⁴/ft.
 Dg = Itr/Avg. Lb = 32273.6/40.00 = 806.84 in⁴/ft.
 Bg = min[Cg (Db/Dg)^{0.25} Lg= 66.57 ft.; 2/3 x 120.00 ft.= 80.00 ft.]
 = 66.57 ft.
 Continuity Factor= 1.0 since Not Continuous
 Wg = 1.0 x (3.271/40.00) x 66.57 x 40.00 = 217.7 Kips

RIGHT GIRDER CALCULATIONS:

Girder section: W36x182 d= 36.330 in. A= 53.60 in.² Ix= 11300 in.⁴
 Same as Left girder.

COMBINED MODE CALCULATIONS:

Using girder with smaller frequency:

δb = 0.260 in. δg = 0.201 in.

System frequency, fn= 0.18 $\sqrt{\frac{386}{\delta b + \delta g}}$ = 5.21 Hz

Wb= 183.1 Kips Wg = 217.7 kips

$$W_c = \frac{\delta b}{\delta b + \delta g} W_b + \frac{\delta g}{\delta b + \delta g} W_g$$

$$= \frac{0.260}{0.461} \times 183.1 + \frac{0.201}{0.461} \times 217.7 = 198.2 \text{ Kips} = 198219 \text{ lbs}$$

ξ = modal damping ratio = 0.01

$$(ap/g) = P_o \exp(-0.35 f_n) / (\xi W_c)$$

$$= 65 \exp(-0.35 \times 5.21) / (0.01 \times 198219)$$

$$= 0.53 \% > 0.50 \% \text{ - DOES NOT SATISFY CRITERION}$$

Since fn= 5.21 Hz ≤ 9 Hz - Stiffness criterion does not need to be checked.

Warning:

The floor system does not satisfy the specified criterion.

-- End --

Appendix A.1 AISC Design Guide 11 Calculations

A.1.5 BRITISH FLOOR SYSTEM

SCI Headquarters

File # BR3

Bay 1

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 02/10/03 By: MDB
Job Id: SCI Headquarters --- Ascot, G.B.
Id: C-D/2-3 Finished

Page 1

Span= 24.442 ft		() Mezzanine	Defaults
Left Girder:		() Beam//OS	Table: FLOORVIB.VDT
Wall		() Girder//OS	Floor Width: 48.885 ft
	Beam	Right Girder:	Floor Length: 78.74 ft
	305X127X42	Wall	() Beam Continuity
	3 Sps @ 97.77"		() Truss or Joist
Left		Right	Girder Continuity
Bm. Span:	Bm Span= 19.685 ft	Bm. Span:	Acc. Limit: 0.5 %
0.0 ft		19.685 ft	Modal Damping: 0.03
Concrete: dc: 5-1/8 in. (total depth)		Activity: WALKING	
fc': 5 Ksi		Occupancy: OFFICE OR RESIDENCE	
wt.: 147 pcf		Design Criteria: AISC Guide, Chapter 4	
Deck Height: 2"		System Evaluation:	
Loading: Dead: 4 psf		SATISFIES CRITERION	
Collateral: 19.36 psf		Combined Mode $a_p/g = 0.21 \% \leq 0.50 \%$	
(V) Live: 11 psf			

VIBRATION ANALYSIS:

Activity: Walking
Occupancy Category: Office or Residence
Evaluation Criterion: Walking, AISC Design Guide, Chapter 4
Reference: Murray, T.M., Allen, D.E. and Ungar, E.E.,
"Floor Vibrations Due To Human Activity",
Design Guide, June 1997

Constant Force, $P_o = 65 \text{ lb}$
Modal Damping Ratio, $\delta = 0.03$
Acceleration Limit, $a_o/g \times 100\% = 0.50 \%$
Joist bottom chords are not extended
Girders are not continuous at columns

	Section	w, plf	Itr, in4	f, Hz.
Beam	305X127X42	727.7	906.1	11.56
Left Wall				
Right Wall	Wall	1789.0	3778.3	9.77
Bay (Using smaller girder frequency)				7.46

Evaluation:

-- Continued --

Appendix A.1.5

AISC Design Guide 11 Calculations

British Floor System (Cont.)

SCI Headquarters

File # BR3

Bay 1

Date: 02/10/03 By: MDB
 Job Id: SCI Headquarters --- Ascot, G.B.
 Id: C-D/2-3 Finished

Page 2

Combined Mode $\Delta_p/g = 0.21 \% \leq 0.50 \%$

The system SATISFIES THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 51.5 psf
 Dead loads = 4.0 psf
 Collateral loads = 19.4 psf
 Live loads = 11.0 psf
 Actual beam and girder weights

Tributary width for left girder = $19.69/2 = 9.84$ ft.

Tributary width for right girder = $2 \times (19.69/2) = 19.69$ ft.

CONCRETE/SLAB DATA:

Concrete	dc= 5.12 in.	fc' = 5 Ksi
	wt= 147 pcf	Ec= 3985 Ksi

Modular ratio, $n = E_s / (1.35 E_c) = 5.39$

Deck height: 2 in.

Effective concrete thickness in deck: 1 in.

SUPPORTED MEMBER CALCULATIONS:

Built-up member: 305X127X42 (28.2 plf)

A= 8.280 in.² I_x= 197 in.⁴ y_c= 6.05 in.

Spacing: S = 97.77 in.

Span: L_b = 19.69 ft.

Uniform load: w_b = $(51.5 + 4.0 + 19.4 + 11.0) \times 97.77/12 + 28.2$
 = 727.7 plf

Transformed moment of inertia:

Effective concrete width = $\min(0.4 L_b, S) = 94.488$ in.

Effective concrete depth = 3.125 in.

Transformed concrete width = 17.528 in.

Transformed concrete area = 54.776 in.²

Distance to neutral axis = 8.350 in. (Above beam c. g.)

Transformed moment of inertia = 906.1 in.⁴

$$\delta_b = \frac{5 w_b L_b^4}{384 E_s I_b} = \frac{5 \times 727.7 \times 19.69^4 \times 1728}{384 \times E_s \times 906.1} = 0.094 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_b} \right]^{0.5}$$

-- Continued --

Appendix A.1.5

AISC Design Guide 11 Calculations

British Floor System (Cont.)

SCI Headquarters
File # BR3
Bay 1

Date: 02/10/03 By: MDB
 Job Id: SCI Headquarters --- Ascot, G.B.
 Id: C-D/2-3 Finished

Page 3

$$= 0.18 \times \left[\frac{386}{0.094} \right]^{0.5} = 11.56 \text{ Hz.}$$

Cj = 2.0

Floor Width = 48.88 ft.

Ds = (12 de³)/(12 n) = (12 x 4.125³)/(12 x 5.39) = 13.02 in⁴/ft.

Db = Itr/S = 906.1/8.15 = 111.21 in⁴/ft.

Bb = min[Cj (Ds/Db)^{0.25} Lb = 23.03 ft.; 2/3 x 48.88 ft. = 32.59 ft.]
 = 23.03 ft.

Continuity Factor = 1.5 since Continuous

and max. adj. Lb = 19.69 ft > 0.7 Lb = 13.78 ft.

Wb = 1.5 x (0.728/8.15) x 23.03 x 19.69 = 60.7 Kips

RIGHT GIRDER CALCULATIONS:

Built-up member: Wall (30.4 plf)

A = 8.94 in.² Ix = 1423 in.⁴ yc = 13.45 in.

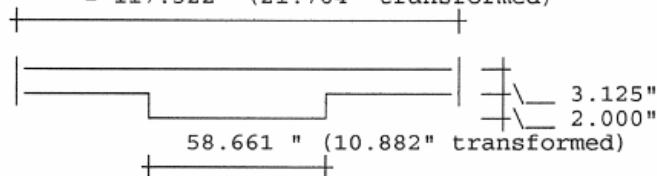
Tributary width = 19.69 ft.

Span: Lg = 24.44 ft.

Equivalent uniform load: wg = 19.69 x (727.7/8.15) + 30.4
 = 1789.0 plf

Transformed moment of inertia:

min(0.2 Lg, 19.69 x 12/2) + min(0.2 Lg, 19.69 x 12/2)
 = 117.322" (21.764" transformed)



Effective concrete width = 117.322 in. and 58.661 in.

Transformed concrete width = 21.764 in. and 10.882 in.

Transformed concrete area = 68.013 in.² and 21.764 in.²

Distance to neutral axis = 14.907 in. (Above girder c. g.)

Transformed moment of inertia = 3778.3 in.⁴

$$\delta_g = \frac{5 \text{ wg Lg}^4}{384 \text{ Es Ig}} = \frac{5 \times 1788. \times 24.44^4 \times 1728}{384 \times \text{Es} \times 3778.30} = 0.131 \text{ in.}$$

$$\text{Frequency} = 0.18 \times \left[\frac{g}{\delta_g} \right]^{0.5}$$

-- Continued --

Appendix A.1.5

AISC Design Guide 11 Calculations

British Floor System (Cont.)

SCI Headquarters

File # BR3

Bay 1

Date: 02/10/03 By: MDB
 Job Id: SCI Headquarters --- Ascot, G.B.
 Id: C-D/2-3 Finished

Page 4

$$= 0.18 \times \left[\frac{386}{0.131} \right]^{0.5} = 9.77 \text{ Hz.}$$

Cg = 1.8

Floor Length= 78.74 ft.

Db = Itr/S = 906.1/8.15 = 111.21 in⁴/ft.

Dg = Itr/Avg. Lb = 3778.3/19.69 = 191.94 in⁴/ft.

Bg = min[Cg (Db/Dg)^{0.25} Lg= 38.38 ft.; 2/3 x 78.74 ft.= 52.49 ft.]
 = 38.38 ft.

Continuity Factor= 1.0 since Not Continuous

Wg = 1.0 x (1.789/19.69) x 38.38 x 24.44 = 85.3 Kips

COMBINED MODE CALCULATIONS:

δb = 0.094 in.

δrg = 0.131 in. (Right girder controls)

$$\text{System frequency, } f_n = 0.18 \sqrt{\frac{386}{\delta b + \delta g}} = 7.46 \text{ Hz}$$

Wb= 60.7 Kips

Wg = 85.3 kips

$$\begin{aligned} W_c &= \frac{\delta b}{\delta b + \delta g} W_b + \frac{\delta g}{\delta b + \delta g} W_g \\ &= \frac{0.094}{0.225} \times 60.7 + \frac{0.131}{0.225} \times 85.3 = 75.0 \text{ Kips} = 75049 \text{ lbs} \end{aligned}$$

ξ = modal damping ratio = 0.03

$$\begin{aligned} (a_p/g) &= P_o \exp(-0.35 f_n) / (\xi W_c) \\ &= 65 \exp(-0.35 \times 7.46) / (0.03 \times 75049) \\ &= 0.21 \% \leq 0.50 \% \quad \text{-- SATISFIES CRITERION} \end{aligned}$$

Since f_n = 7.46 Hz ≤ 9 Hz - Stiffness criterion does not need to be checked.

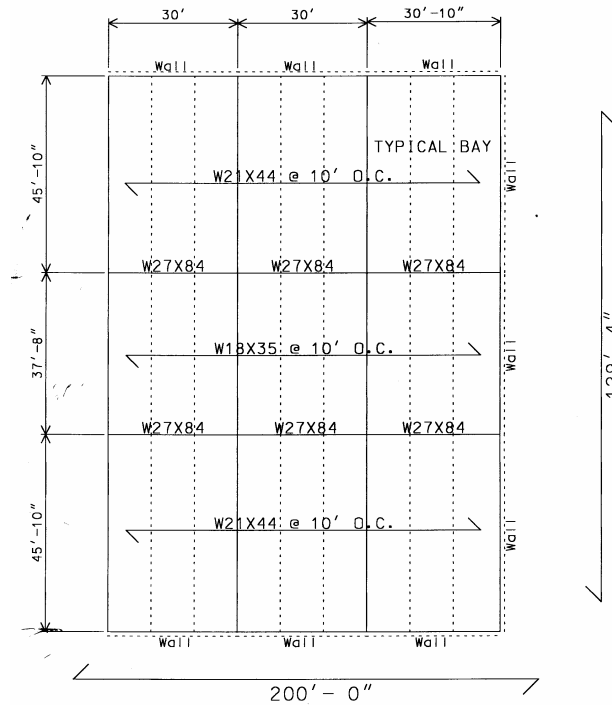
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APPENDIX A.2 SCI FREQUENCY CALCULATIONS

Appendix A.2 SCI Frequency Calculations

A.2.1 HOT-ROLLED SHAPES

Newport V
File # 596
Typical Bay



Loading and Damping

DL := 4 psf

CL := 12.5 psf

LL := 8 psf

Note : The collateral load accounts for raised flooring and is not indicative of most offices.

Damping : $\zeta := 2.5 \%$

Deck Properties

Compressive Strength :

$f_c := 3.5$ ksi

Concrete : $w_c := 115$ pcf

Total Floor Thickness :

$t_{conc} := 5.25$ in.

Deck Height : $d_h := 2$ in.

Slab/Deck Weight :

$d_w := 41.7$ psf

Beam Properties

Main Beam

W21X44

$w_{mb} := 44.0$ lb./ft

$A_{mb} := 13.0$ in.²

$I_{x_mb} := 843$ in.⁴

e.n.a. $y_{c_mb} := 10.33$ in.

$L_{mb} := 45.83$ ft

Adjacent Beam

W18X35

$w_{ab} := 35.0$ lb./ft

$A_{ab} := 10.3$ in.²

$I_{x_ab} := 510$ in.⁴

e.n.a. $y_{c_ab} := 8.85$ in.

$L_{ab} := 37.67$ ft

Girder Properties

Main Girder

W27X84

$w_{mg} := 84.0$ lb./ft

$A_{mg} := 24.8$ in.²

$I_{x_mg} := 2850$ in.⁴

e.n.a. $y_{c_mg} := 13.36$ in.

$L_{mg} := 30.83$ ft

Adjacent Girder

W27X84

$w_{ag} := 84.0$ lb./ft

$A_{ag} := 24.8$ in.²

$I_{x_ag} := 2850$ in.⁴

e.n.a. $y_{c_ag} := 13.36$ in.

$L_{ag} := 30.00$ ft

Note : The elastic neutral axis (e.n.a) is taken from the top of the beam, joist, or girder

Appendix A.2.1 SCI Frequency Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Secondary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between $L_{mb}/4 = 11' - 5.5"$ and the beam spacing of $10' - 3"$.

$$\text{Therefore, } b_{\text{eff}_b} := \frac{L_{mg}}{3} \quad b_{\text{eff}_b} = 10.28 \text{ ft}$$

For the beam only the concrete above the steel form deck and a dynamic concrete modulus of elasticity of $1.35E_c$ are used in the calculations.

$$E_s := 29000 \text{ ksi}$$

$$E_c := w_c^{1.5} \cdot \sqrt{f_c} \quad E_c = 2307 \text{ ksi}$$

$$\text{Modular Ratio } n := \frac{E_s}{1.35 E_c} \quad n = 9.31$$

$$y_{\text{bar}} := \frac{\left[(A_{mb}) \cdot (d_h + y_{c_mb}) - \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)^2 \cdot \frac{1}{2} \right]}{A_{mb} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)} \quad y_{\text{bar}} = 1.61 \text{ in.}$$

below top of form deck

$$I_{b_comp} := I_{x_mb} + A_{mb} \cdot (d_h + y_{c_mb} - y_{\text{bar}})^2 + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot \frac{(t_{\text{conc}} - d_h)^3}{12} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h) \cdot \left[y_{\text{bar}} + \frac{(t_{\text{conc}} - d_h)}{2} \right]^2$$

$$I_{b_comp} = 2825.32$$

Primary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between

$$\text{Girder Spacing: } Sp := \frac{L_{mb} + L_{ab}}{2} \quad Sp = 41.75 \text{ ft} \quad \text{and} \quad \frac{L_{mg}}{4} = 7.71 \text{ ft}$$

$$\text{Therefore, } b_{\text{eff}_g} := \frac{L_{mg}}{4} \quad b_{\text{eff}_g} = 7.71 \text{ ft}$$

For the girder the concrete above the steel form deck and one half the deck height and a dynamic concrete modulus of elasticity of $1.35E_c$ are used.

$$y_{\text{bar}} := \frac{\left[(A_{mg}) \cdot \left(\frac{d_h}{2} + y_{c_mg} \right) - \left(\frac{b_{\text{eff}_g} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right)^2 \cdot \frac{1}{2} \right]}{A_{mg} + \left(\frac{b_{\text{eff}_g} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right)} \quad y_{\text{bar}} = 3.98 \text{ in.}$$

below effective slab

Appendix A.2.1 SCI Frequency Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

$$I_{g_comp} := I_{x_mg} + A_{mg} \cdot \left(\frac{d_h}{2} + y_{c_mg} - y_{bar} \right)^2 + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \frac{\left(t_{conc} - \frac{d_h}{2} \right)^3}{12} + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \left(t_{conc} - \frac{d_h}{2} \right) \cdot \left[y_{bar} + \frac{\left(t_{conc} - \frac{d_h}{2} \right)}{2} \right]^2$$

$$I_{g_comp} = 7159.12 \text{ in.}^4$$

MODE SHAPES AND BEAM/GIRDER BOUNDARY CONDITIONS

The fundamental mode is that which gives the lowest frequency and subsequently the highest deflection

Mode A

Uniform Distributed Loading : $w_j := b_{eff_b} \cdot \left(DL + CL + LL + d_w + \frac{w_{mb}}{b_{eff_b}} \right)$ $w_j = 724.32 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (pin-pin boundary conditions)

$$\delta_b := \frac{5 \cdot w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 I_{b_comp}} \quad \delta_b = 0.877 \text{ in.}$$

To account for the stiffening effect offered by the shorter adjacent span, the above deflection needs to be modified. Since the bay is in the corner, only two continuous spans are considered for deflection reduction. Therefore, Equation 3.6 from the AISC publication *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) is used to reduce the deflection of the floor

Main Beam Properties

$$k_m := \frac{I_{b_comp}}{L_{mb} \cdot 12} \quad k_m = 5.14 \text{ in.}^3$$

Adjacent Beam Properties

$$b_{eff_ab} := \frac{L_{ab}}{4} \quad b_{eff_ab} = 9.42 \text{ ft}$$

$$y_{bar} := \frac{\left[(A_{ab}) \cdot (d_h + y_{c_ab}) - \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot (t_{conc} - d_h)^2 \cdot \frac{1}{2} \right]}{A_{ab} + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot (t_{conc} - d_h)} \quad y_{bar} = 0.96 \text{ in.}$$

below top of form deck

$$I_{ab_comp} := I_{x_ab} + A_{ab} \cdot (d_h + y_{c_ab} - y_{bar})^2 + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot \frac{(t_{conc} - d_h)^3}{12} + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot (t_{conc} - d_h) \cdot \left[y_{bar} + \frac{(t_{conc} - d_h)}{2} \right]^2$$

$$I_{ab_comp} = 1815.78 \text{ in.}^4$$

Appendix A.2.1 SCI Frequency Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

$$k_s := \frac{I_{ab_comp}}{L_{ab} \cdot 12} \quad k_s = 4.02 \text{ in.}^3$$

$$\Delta_b := \left[\frac{0.4 + \frac{k_m}{k_s} \cdot \left(1 + 0.6 \cdot \frac{L_{ab}^2}{L_{mb}^2} \right)}{1 + \frac{k_m}{k_s}} \right] \cdot \delta_b \quad \Delta_b = 0.846$$

Natural Frequency for Mode A :

$$f_A := 0.18 \cdot \sqrt{\frac{386}{\Delta_b}} \quad f_A = 3.84 \text{ Hz}$$

Mode B

Uniform Distributed Loading : $w_g := \left(\frac{L_{mb} + L_{ab}}{2} \right) \cdot \left(\frac{w_j}{b_{eff_b}} \right) + w_{mg} \quad w_g = 3027 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (fixed-ended boundary conditions)

$$\delta_b := \frac{w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_comp}} \quad \delta_b = 0.175 \text{ in.}$$

Girder Deflection : Primary Framing (pin-pin boundary conditions)

$$\delta_g := \frac{5 \cdot w_g \cdot L_{mg}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{g_comp}} \quad \delta_g = 0.296 \text{ in.}$$

To account for the stiffening effect offered by the shorter adjacent span, the above deflection needs to be modified. Since the bay is in the corner, only two continuous spans are considered for deflection reduction. Therefore, Equation 3.6 from the AISC publication *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) is used to reduce the deflection of the floor

Main Girder Properties

$$k_m := \frac{I_{g_comp}}{L_{mg} \cdot 12} \quad k_m = 19.35 \text{ in.}^3$$

Appendix A.2.1 SCI Frequency Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Adjacent Girder Properties

$$k_s := \frac{I_{g_comp}}{L_{ag} \cdot 12} \quad k_s = 19.89 \text{ in.}^3$$

$$\Delta_g := \left[\frac{0.4 + \frac{k_m}{k_s} \cdot \left(1 + 0.6 \frac{L_{ag}^2}{L_{mg}^2} \right)}{1 + \frac{k_m}{k_s}} \right] \cdot \delta_g \quad \Delta_g = 0.289$$

Natural Frequency for Mode B :

$$f_B := 0.18 \sqrt{\frac{386}{\delta_b + \Delta_g}} \quad f_B = 5.19 \text{ Hz}$$

$f_B > f_A$, therefore, Mode A controls the motion of the Floor and the corresponding Natural Frequency is 3.84 Hz.

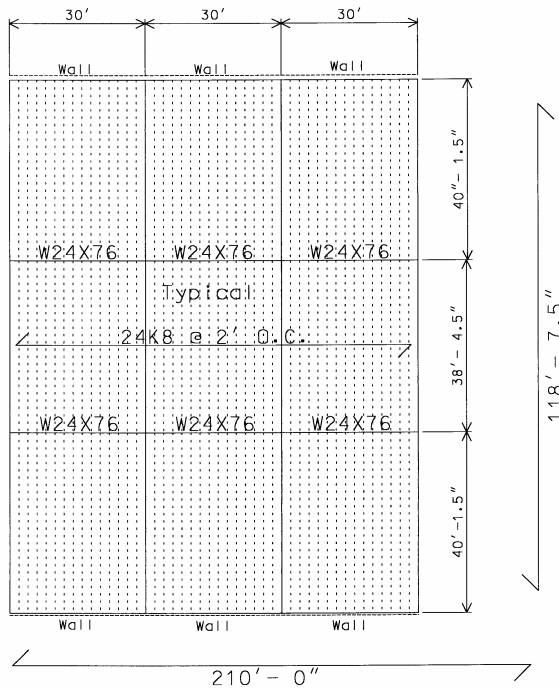
Appendix A.2 SCI Frequency Calculations

A.2.2 HOT-ROLLED GIRDERS WITH JOISTS

Melroe Corporation

File # 563

Typical Bay



Loading and Damping

DL := 7 psf

CL := 0 psf

LL := 11 psf

Damping : $\zeta := 3.0 \%$

Deck Properties

Compressive Strength :

$f_c := 3.0$ ksi

Concrete : $w_c := 145$ pcf

Total Floor Thickness :

$t_{conc} := 2.75$ in.

Deck Height : $d_h := 0.625$ in.

Slab/Deck Weight :

$d_w := 30.5$ psf

Beam Properties

Main Beam

24K8

$w_{mb} := 11.5$ lb./ft

$A_{mb} := 2.157$ in.²

$I_{x_mb} := 282$ in.⁴

e.n.a. $y_{c_mb} := 10.72$ in.

$L_{mb} := 38.375$ ft

Depth $D_{mb} := 24$ in.

Adjacent Beam

24K8

$w_{ab} := 11.5$ lb./ft

$A_{ab} := 2.157$ in.²

$I_{x_ab} := 282$ in.⁴

e.n.a. $y_{c_ab} := 10.72$ in.

$L_{ab} := 40.125$ ft

Depth $D_{mb} := 24$ in.

Girder Properties

Main Girder

W24X76

$w_{mg} := 76.0$ lb./ft

$A_{mg} := 22.40$ in.²

$I_{x_mg} := 2100$ in.⁴

e.n.a. $y_{c_mg} := 11.96$ in.

$L_{mg} := 30.00$ ft

Adjacent Girder

W24X76

$w_{ag} := 76.0$ lb./ft

$A_{ag} := 22.40$ in.²

$I_{x_ag} := 2100$ in.⁴

e.n.a. $y_{c_ag} := 11.96$ in.

$L_{ag} := 30.00$ ft

Note : The elastic neutral axis (e.n.a) is taken from the top of the beam, joist, or girder

Appendix A.2.2 SCI Frequency Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Secondary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between $L_{mb}/4 = 9' - 7"$ and the beam spacing of $2' - 0"$.

$$\text{Therefore, } b_{\text{eff}_b} := \frac{L_{mb}}{15} \quad b_{\text{eff}_b} = 2 \quad \text{ft}$$

For the beam only the concrete above the steel form deck and a dynamic concrete modulus of elasticity of $1.35E_c$ are used in the calculations.

$$E_s := 29000 \quad \text{ksi}$$

$$E_c := w_c^{1.5} \cdot \sqrt{f_c} \quad E_c = 3024 \quad \text{ksi}$$

$$\text{Modular Ratio } n := \frac{E_s}{1.35 \cdot E_c} \quad n = 7.1$$

$$y_{\text{bar}} := \frac{\left[(A_{mb}) \cdot (d_h + y_{c_mb}) - \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)^2 \cdot \frac{1}{2} \right]}{A_{mb} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)} \quad y_{\text{bar}} = 1.8 \quad \text{in. below top of form deck}$$

$$I_{b_comp} := I_{x_mb} + A_{mb} \cdot (d_h + y_{c_mb} - y_{\text{bar}})^2 + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot \frac{(t_{\text{conc}} - d_h)^3}{12} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h) \cdot \left[y_{\text{bar}} + \frac{(t_{\text{conc}} - d_h)}{2} \right]^2$$

$$I_{b_comp} = 540.05 \quad \text{in.}^4$$

Since the secondary framing member is a joist, shear deformations due to the web are taken into account and the above composite moment of inertia needs to be reduced. Equations from chapter 3 of the AISC Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) are used to reduce the moment of inertia for open web joists.

$$C_r := 0.9 \cdot \left[1 - e^{-.28 \cdot \left(12 \cdot \frac{L_{mb}}{D_{mb}} \right)} \right]^{2.8} \quad C_r = 0.89 \quad (\text{Equation 3.16, AISC Design Guide})$$

$$\gamma := \frac{1}{C_r} - 1 \quad \gamma = 0.13 \quad (\text{Equation 3.19, AISC Design Guide})$$

$$I_{b_eff} := \frac{1}{\frac{\gamma}{I_{x_mb}} + \frac{1}{I_{b_comp}}} \quad I_{b_eff} = 435.28 \quad \text{in.}^4 \quad (\text{Equation 3.18, AISC Design Guide})$$

Appendix A.2.2 SCI Frequency Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Primary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between

$$\text{Girder Spacing : } S_p := \frac{L_{mb} + L_{ab}}{2} \quad S_p = 39.25 \text{ ft} \quad \text{and} \quad \frac{L_{mg}}{4} = 7.5 \text{ ft}$$

$$\text{Therefore, } b_{eff_g} := \frac{L_{mg}}{4} \quad b_{eff_g} = 7.5 \text{ ft}$$

For the girder the concrete above the steel form deck and one half the deck height and a dynamic concrete modulus of elasticity of $1.35E_c$ are used.

$$y_{bar} := \frac{\left[(A_{mg}) \cdot \left(\frac{d_h}{2} + y_{c_mg} \right) - \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \left(t_{conc} - \frac{d_h}{2} \right) \cdot \frac{1}{2} \right]}{A_{mg} + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \left(t_{conc} - \frac{d_h}{2} \right)} \quad y_{bar} = 4.45 \text{ in. below effective slab}$$

$$I_{g_comp} := I_{x_mg} + A_{mg} \cdot \left(\frac{d_h}{2} + y_{c_mg} - y_{bar} \right)^2 + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \frac{\left(t_{conc} - \frac{d_h}{2} \right)^3}{12} + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \left(t_{conc} - \frac{d_h}{2} \right) \cdot \left[y_{bar} + \frac{\left(t_{conc} - \frac{d_h}{2} \right)}{2} \right]^2$$

$$I_{g_comp} = 4478.43 \text{ in.}^4$$

Since the secondary framing member is a joist, the above girder composite moment of inertia needs to be reduced due to lack of stiffness of the joist seats. Equations from chapter 3 of the AISC Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) are used to reduce the moment of inertia.

$$I_{g_eff} := I_{x_mg} + \frac{(I_{g_comp} - I_{x_mg})}{4} \quad I_{g_eff} = 2694.61 \text{ in.}^4 \quad (\text{Equation 3.14, AISC Design Guide})$$

MODE SHAPES AND BEAM/GIRDER BOUNDARY CONDITIONS

The fundamental mode is that which gives the lowest frequency and subsequently the highest deflection

Mode A

$$\text{Uniform Distributed Loading : } w_j := b_{eff_b} \cdot \left(DL + CL + LL + d_w + \frac{w_{mb}}{b_{eff_b}} \right) \quad w_j = 108.5 \text{ lb./ft}$$

Appendix A.2.2 SCI Frequency Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation File # 563 Typical Bay

Beam/Joist Deflection : Secondary Framing (pin-pin boundary conditions)

$$\delta_b := \frac{5 \cdot w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_eff}} \quad \delta_b = 0.419 \text{ in.}$$

Joist framing lacks continuity between adjacent beams. Therefore, the above deflection needs does not need to be modified.

Natural Frequency for Mode A :

$$f_A := 0.18 \cdot \sqrt{\frac{386.4}{\delta_b}} \quad f_A = 5.46 \text{ Hz}$$

Mode B

Uniform Distributed Loading : $w_g := \left(\frac{L_{mb} + L_{ab}}{2} \right) \cdot \left(\frac{w_j}{b_{eff_b}} \right) + w_{mg} \quad w_g = 2205 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (fixed-ended boundary conditions)

$$\delta_b := \frac{w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_eff}} \quad \delta_b = 0.084 \text{ in.}$$

Girder Deflection : Primary Framing (pin-pin boundary conditions)

$$\delta_g := \frac{5 \cdot w_g \cdot L_{mg}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{g_eff}} \quad \delta_g = 0.514 \text{ in.}$$

To account for the stiffening effect offered by the adjacent span, the above deflection needs to be modified. Therefore, Equation 3.7 from the AISC publication *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) is used to reduce the deflection of the floor for three continuous spans .

Main Girder Properties

$$k_m := \frac{I_{g_eff}}{L_{mg} \cdot 12} \quad k_m = 7.49 \text{ in.}^3$$

Adjacent Girder Properties

$$k_s := \frac{I_{g_eff}}{L_{ag} \cdot 12} \quad k_s = 7.49 \text{ in.}^3$$

Appendix A.2.2
SCI Frequency Calculations
Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

$$\Delta_g := \left[\frac{0.6 + 2 \cdot \frac{k_m}{k_s} \cdot \left(1 + 1.2 \cdot \frac{L_{ag}^2}{L_{mg}^2} \right)}{3 + 2 \cdot \frac{k_m}{k_s}} \right] \cdot \delta_g \quad \Delta_g = 0.514$$

Natural Frequency for Mode B :

$$f_B := 0.18 \sqrt{\frac{386.4}{\delta_b + \Delta_g}} \quad f_B = 4.57 \text{ Hz}$$

$f_A > f_B$, therefore, Mode B controls the motion of the Floor and the corresponding Natural Frequency is 4.57 Hz

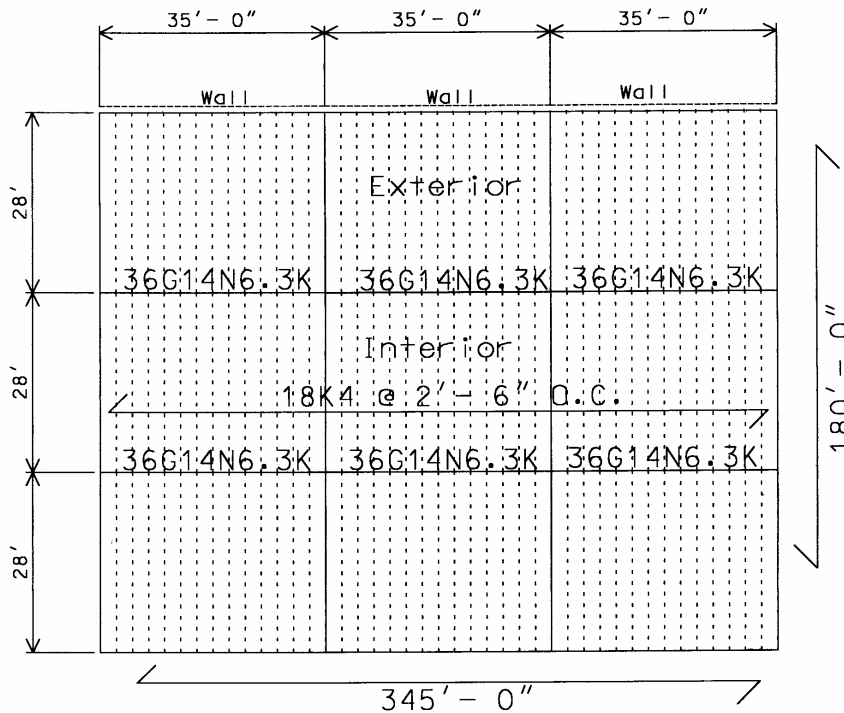
Appendix A.2 SCI Frequency Calculations

A.2.3 JOIST GIRDERS WITH JOISTS

Garmin International

File # 506

Typical Exterior Bay



Loading and Damping

DL := 4 psf

CL := 0 psf

LL := 11 psf

Damping : $\zeta := 2.5 \%$

Deck Properties

Compressive Strength :

$f_c := 3.5$ ksi

Concrete : $w_c := 150$ pcf

Total Floor Thickness :

$t_{conc} := 2.50$ in.

Deck Height : $d_h := 0.625$ in.

Slab/Deck Weight :

$d_w := 28.3$ psf

Beam Properties

Main Beam

18K4

$w_{mb} := 7.2$ lb./ft

$A_{mb} := 1.431$ in.²

$I_{x_mb} := 102$ in.⁴

$y_{c_mb} := 8.06$ in.

$L_{mb} := 28.00$ ft

Depth $D_{mb} := 18$ in.

Adjacent Beam

18K4

$w_{ab} := 7.2$ lb./ft

$A_{ab} := 1.431$ in.²

$I_{x_ab} := 102$ in.⁴

$y_{c_ab} := 8.06$ in.

$L_{ab} := 28.00$ ft

Depth $D_{ab} := 18$ in.

Girder Properties

Main Girder

36G14N6.3K

$w_{mg} := 27.1$ lb./ft

$A_{mg} := 6.38$ in.²

$I_{x_mg} := 1929$ in.⁴

e.n.a. $y_{c_mg} := 16.06$ in.

$L_{mg} := 35.00$ ft

Depth $D_{mg} := 36$ in.

Adjacent Girder

36G14N6.3K

$w_{ag} := 27.1$ lb./ft

$A_{ag} := 6.38$ in.²

$I_{x_ag} := 1929$ in.⁴

e.n.a. $y_{c_ag} := 16.06$ in.

$L_{ag} := 35.00$ ft

Depth $D_{mg} := 36$ in.

Note : The elastic neutral axis (e.n.a) is taken from the top of the beam, joist, or girder

Appendix A.2.3 SCI Frequency Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Secondary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between $L_{mb}/4 = 7' - 0"$ and the beam spacing of $2' - 6"$.

$$\text{Therefore, } b_{\text{eff}_b} := \frac{L_{mb}}{14} \quad b_{\text{eff}_b} = 2.5 \quad \text{ft}$$

For the beam only the concrete above the steel form deck and a dynamic concrete modulus of elasticity of $1.35E_c$ are used in the calculations.

$$E_s := 29000 \quad \text{ksi}$$

$$E_c := w_c^{1.5} \cdot \sqrt{f_c} \quad E_c = 3437 \quad \text{ksi}$$

$$\text{Modular Ratio } n := \frac{E_s}{1.35 \cdot E_c} \quad n = 6.25$$

$$y_{\text{bar}} := \frac{\left[(A_{mb}) \cdot (d_h + y_{c_mb}) - \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)^2 \cdot \frac{1}{2} \right]}{A_{mb} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)} \quad y_{\text{bar}} = 0.38 \quad \text{in.}$$

below top of form deck

$$I_{b_comp} := I_{x_mb} + A_{mb} \cdot (d_h + y_{c_mb} - y_{\text{bar}})^2 + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot \frac{(t_{\text{conc}} - d_h)^3}{12} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h) \cdot \left[y_{\text{bar}} + \frac{(t_{\text{conc}} - d_h)}{2} \right]^2$$

$$I_{b_comp} = 218.96 \quad \text{in.}^4$$

Since the secondary framing member is a joist, shear deformations due to the web are taken into account and the above composite moment of inertia needs to be reduced. Equations from chapter 3 of the AISC Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) are used to reduce the moment of inertia for open web joists.

$$C_r := 0.9 \cdot \left[1 - e^{-.28 \cdot \left(12 \cdot \frac{L_{mb}}{D_{mb}} \right)} \right]^{2.8} \quad C_r = 0.89 \quad (\text{Equation 3.16, AISC Design Guide})$$

$$\gamma := \frac{1}{C_r} - 1 \quad \gamma = 0.13 \quad (\text{Equation 3.19, AISC Design Guide})$$

$$I_{b_eff} := \frac{1}{\frac{\gamma}{I_{x_mb}} + \frac{1}{I_{b_comp}}} \quad I_{b_eff} = 171.76 \quad \text{in.}^4 \quad (\text{Equation 3.18, AISC Design Guide})$$

Appendix A.2.3 SCI Frequency Calculations Joist Girders with Joists (Cont.)

**Garmin International
File # 506
Typical Exterior Bay**

Primary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between

$$\text{Girder Spacing: } Sp := \frac{L_{mb} + L_{ab}}{2} \quad Sp = 28 \quad \text{ft} \quad \text{and} \quad \frac{L_{mg}}{4} = 8.75 \quad \text{ft}$$

$$\text{Therefore, } b_{\text{eff_g}} := \frac{L_{mg}}{4} \quad b_{\text{eff_g}} = 8.75 \quad \text{ft}$$

For the girder the concrete above the steel form deck and one half the deck height and a dynamic concrete modulus of elasticity of $1.35E_c$ are used.

$$y_{\text{bar}} := \frac{\left[(A_{mg}) \cdot \left(\frac{d_h}{2} + y_{c_mg} \right) - \left(\frac{b_{\text{eff_g}} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right) \cdot \frac{1}{2} \right]}{A_{mg} + \left(\frac{b_{\text{eff_g}} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right)} \quad y_{\text{bar}} = 1.49 \quad \text{in. below effective slab}$$

$$I_{g_comp} := I_{x_mg} + A_{mg} \cdot \left(\frac{d_h}{2} + y_{c_mg} - y_{\text{bar}} \right)^2 + \left(\frac{b_{\text{eff_g}} \cdot 12}{n} \right) \cdot \frac{\left(t_{\text{conc}} - \frac{d_h}{2} \right)^3}{12} + \left(\frac{b_{\text{eff_g}} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right) \cdot \left[y_{\text{bar}} + \frac{\left(t_{\text{conc}} - \frac{d_h}{2} \right)}{2} \right]^2$$

$$I_{g_comp} = 3602.08 \quad \text{in.}^4$$

Since the primary framing member is a joist girder, shear deformations due to the web are taken into account and the above composite moment of inertia needs to be reduced.

Equations from chapter 3 of the AISC Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) are used to reduce the moment of inertia for open web joists.

$$C_r := 0.9 \cdot \left[1 - e^{-.28 \cdot \left(12 \cdot \frac{L_{mg}}{D_{mg}} \right)} \right]^{2.8} \quad C_r = 0.81 \quad (\text{Equation 3.16, AISC Design Guide})$$

$$\gamma := \frac{1}{C_r} - 1 \quad \gamma = 0.24 \quad (\text{Equation 3.19, AISC Design Guide})$$

$$I_{g_eff} := \frac{1}{\frac{\gamma}{I_{x_mg}} + \frac{1}{I_{g_comp}}} \quad I_{g_eff} = 2490.89 \quad \text{in.}^4 \quad (\text{Equation 3.18, AISC Design Guide})$$

Appendix A.2.3 SCI Frequency Calculations Joist Girders with Joists (Cont.)

Garmin International File # 506 Typical Exterior Bay

Since the secondary framing member is a joist, the above girder effective composite moment of inertia needs to be reduced due to lack of stiffness of the joist seats. Equations from chapter 3 of the AISC Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) are used to reduce the moment of inertia.

$$I_{g_fin} := I_{x_mg} + \frac{(I_{g_eff} - I_{x_mg})}{4} \quad I_{g_fin} = 2069.47 \text{ in.}^4 \quad (\text{Equation 3.14, AISC Design Guide})$$

MODE SHAPES AND BEAM/GIRDER BOUNDARY CONDITIONS

The fundamental mode is that which gives the lowest frequency and subsequently the highest deflection

Mode A

Uniform Distributed Loading : $w_j := b_{eff_b} \cdot \left(DL + CL + LL + d_w + \frac{w_{mb}}{b_{eff_b}} \right) \quad w_j = 115.5 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (pin-pin boundary conditions)

$$\delta_b := \frac{5 \cdot w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_eff}} \quad \delta_b = 0.321 \text{ in.}$$

Joist framing lacks continuity between adjacent beams. Therefore, the above deflection does not need to be modified.

Natural Frequency for Mode A :

$$f_A := 0.18 \cdot \sqrt{\frac{386.4}{\delta_b}} \quad f_A = 6.25 \text{ Hz}$$

Mode B

Uniform Distributed Loading : $w_g := \left(\frac{L_{mb} + L_{ab}}{2} \right) \cdot \left(\frac{w_j}{b_{eff_b}} \right) + w_{mg} \quad w_g = 1320 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (fixed-ended boundary conditions)

$$\delta_b := \frac{w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_eff}} \quad \delta_b = 0.064 \text{ in.}$$

Appendix A.2.3 SCI Frequency Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Girder Deflection : Primary Framing (pin-pin boundary conditions)

$$\delta_g := \frac{5 \cdot w_g \cdot L_{mg}^4 \cdot 1728}{384 E_s \cdot 1000 I_{g_fin}} \quad \delta_g = 0.743 \text{ in.}$$

Joist framing lacks continuity between adjacent beams. Therefore, the above deflection does not need to be modified.

Natural Frequency for Mode B :

$$f_B := 0.18 \sqrt{\frac{386.4}{\delta_b + \delta_g}} \quad f_B = 3.94 \text{ Hz}$$

$f_A > f_B$, therefore, Mode B controls the motion of the Floor and the corresponding Natural Frequency is 3.94 Hz.

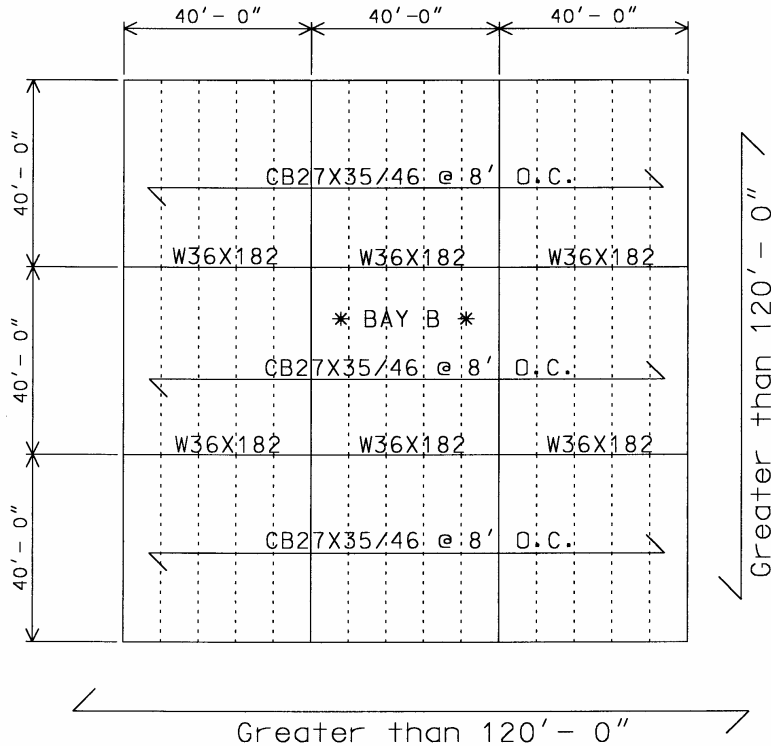
Appendix A.2 SCI Frequency Calculations

A.2.4 HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS

AMAT Building 34

File # Boice7

Bay "B"



Loading and Damping

DL := 0 psf

CL := 0 psf

LL := 0 psf

Damping : $\zeta := 1.0 \%$

Deck Properties

Compressive Strength :

$f_c := 4.0$ ksi

Concrete : $w_c := 145$ pcf

Total Floor Thickness :

$t_{conc} := 7.00$ in.

Deck Height : $d_h := 2$ in.

Slab/Deck Weight :

$d_w := 73.5$ psf

Beam Properties

Main Beam

CB27X35/46

$w_{mb} := 29.8$ lb./ft

$A_{mb} := 8.75$ in²

$I_{x_mb} := 1407$ in⁴

e.n.a. $y_{c_mb} := 15.50$ in.

$L_{mb} := 40.00$ ft

Adjacent Beam

CB27X35/46

$w_{ab} := 29.8$ lb./ft

$A_{ab} := 8.75$ in²

$I_{x_ab} := 1407$ in⁴

e.n.a. $y_{c_ab} := 15.50$ in.

$L_{ab} := 40.00$ ft

Girder Properties

Main Girder

W36X182

$w_{mg} := 182.0$ lb./ft

$A_{mg} := 53.60$ in²

$I_{x_mg} := 11300$ in⁴

e.n.a. $y_{c_mg} := 18.17$ in.

$L_{mg} := 40.00$ ft

Adjacent Girder

W36X182

$w_{ag} := 182.0$ lb./ft

$A_{ag} := 53.60$ in²

$I_{x_ag} := 11300$ in⁴

e.n.a. $y_{c_ag} := 18.17$ in.

$L_{ag} := 40.00$ ft

Note :The elastic neutral axis (e.n.a) is taken from the top of the beam, joist, or girder

Appendix A.2.4 SCI Frequency Calculations Hot-Rolled Girders with Castelled Beams (Cont.)

**AMAT Building 34
File # Boice7
Bay "B"**

Secondary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between $L_{mb}/4 = 10' - 0"$ and the beam spacing of $8' - 0"$.

$$\text{Therefore, } b_{\text{eff}_b} := \frac{L_{\text{mg}}}{5} \quad b_{\text{eff}_b} = 8 \quad \text{ft}$$

For the beam only the concrete above the steel form deck and a dynamic concrete modulus of elasticity of $1.35E_c$ are used in the calculations.

$$E_s := 29000 \quad \text{ksi}$$

$$E_c := w_c^{1.5} \cdot \sqrt{f_c} \quad E_c = 3492 \quad \text{ksi}$$

$$\text{Modular Ratio } n := \frac{E_s}{1.35 E_c} \quad n = 6.15$$

$$y_{\text{bar}} := \frac{\left[(A_{\text{mb}}) \cdot (d_h + y_{\text{c_mb}}) - \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)^2 \cdot \frac{1}{2} \right]}{A_{\text{mb}} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h)} \quad y_{\text{bar}} = -0.48 \quad \text{in.}$$

below top of form deck

$$I_{b_comp} := I_{x_mb} + A_{\text{mb}} \cdot (d_h + y_{\text{c_mb}} - y_{\text{bar}})^2 + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot \frac{(t_{\text{conc}} - d_h)^3}{12} + \left(\frac{b_{\text{eff}_b} \cdot 12}{n} \right) \cdot (t_{\text{conc}} - d_h) \cdot \left[y_{\text{bar}} + \frac{(t_{\text{conc}} - d_h)}{2} \right]^2$$

$$I_{b_comp} = 4716.66$$

Primary Framing Composite Properties

For the typical corner bay, the effective concrete slab width is the minimum value between

$$\text{Girder Spacing: } Sp := \frac{L_{mb} + L_{ab}}{2} \quad Sp = 40 \quad \text{ft} \quad \text{and} \quad \frac{L_{\text{mg}}}{4} = 10 \quad \text{ft}$$

$$\text{Therefore, } b_{\text{eff}_g} := \frac{L_{\text{mg}}}{4} \quad b_{\text{eff}_g} = 10 \quad \text{ft}$$

For the girder the concrete above the steel form deck and one half the deck height and a dynamic concrete modulus of elasticity of $1.35E_c$ are used.

$$y_{\text{bar}} := \frac{\left[(A_{\text{mg}}) \cdot \left(\frac{d_h}{2} + y_{\text{c_mg}} \right) - \left(\frac{b_{\text{eff}_g} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right)^2 \cdot \frac{1}{2} \right]}{A_{\text{mg}} + \left(\frac{b_{\text{eff}_g} \cdot 12}{n} \right) \cdot \left(t_{\text{conc}} - \frac{d_h}{2} \right)} \quad y_{\text{bar}} = 3.96 \quad \text{in.}$$

below effective slab

Appendix A.2.4

SCI Frequency Calculations

Hot-Rolled Girders with Castelled Beams (Cont.)

AMAT Building 34

File # Boice7

Bay "B"

$$I_{g_comp} := I_{x_mg} + A_{mg} \cdot \left(\frac{d_h}{2} + y_{c_mg} - y_{bar} \right)^2 + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \frac{\left(t_{conc} - \frac{d_h}{2} \right)^3}{12} + \left(\frac{b_{eff_g} \cdot 12}{n} \right) \cdot \left(t_{conc} - \frac{d_h}{2} \right) \cdot \left[y_{bar} + \frac{\left(t_{conc} - \frac{d_h}{2} \right)}{2} \right]^2$$

$$I_{g_comp} = 29720.99 \text{ in}^4$$

MODE SHAPES AND BEAM/GIRDER BOUNDARY CONDITIONS

The fundamental mode is that which gives the lowest frequency and subsequently the highest deflection.

Mode A

Uniform Distributed Loading : $w_j := b_{eff_b} \cdot \left(DL + CL + LL + d_w + \frac{w_{mb}}{b_{eff_b}} \right)$ $w_j = 617.8 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (pin-pin boundary conditions)

$$\delta_b := \frac{5 \cdot w_j \cdot L_{mb}^4 \cdot 1.728}{384 E_s \cdot 1000 \cdot I_{b_comp}} \quad \delta_b = 0.26 \text{ in.}$$

To account for the stiffening effect offered by the adjacent span, the above deflection needs to be modified. Therefore, Equation 3.7 from the AISC publication *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) is used to reduce the deflection of the floor for three continuous spans.

Main Beam Properties

$$k_m := \frac{I_{b_comp}}{L_{mb} \cdot 12} \quad k_m = 9.83 \text{ in}^3$$

Adjacent Beam Properties

$$b_{eff_ab} := \frac{L_{mg}}{5} \quad b_{eff_ab} = 8 \text{ ft}$$

$$y_{bar} := \frac{\left[(A_{ab}) \cdot \left(d_h + y_{c_ab} \right) - \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot \left(t_{conc} - d_h \right)^2 \cdot \frac{1}{2} \right]}{A_{ab} + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot \left(t_{conc} - d_h \right)} \quad y_{bar} = -0.48 \text{ in.}$$

below top of form deck

$$I_{ab_comp} := I_{x_ab} + A_{ab} \cdot \left(d_h + y_{c_ab} - y_{bar} \right)^2 + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot \frac{\left(t_{conc} - d_h \right)^3}{12} + \left(\frac{b_{eff_ab} \cdot 12}{n} \right) \cdot \left(t_{conc} - d_h \right) \cdot \left[y_{bar} + \frac{\left(t_{conc} - d_h \right)}{2} \right]^2$$

$$I_{ab_comp} = 4716.66 \text{ in}^4$$

Appendix A.2.4

SCI Frequency Calculations

Hot-Rolled Girders with Castellated Beams (Cont.)

AMAT Building 34
File # Boice7
Bay "B"

$$k_s := \frac{I_{ab_comp}}{L_{ab} \cdot 12} \quad k_s = 9.83 \text{ in.}^3$$

$$\Delta_b := \left[\frac{0.6 + 2 \cdot \frac{k_m}{k_s} \cdot \left(1 + 1.2 \cdot \frac{L_{ab}^2}{L_{mb}^2} \right)}{3 + 2 \cdot \frac{k_m}{k_s}} \right] \cdot \delta_b \quad \Delta_b = 0.26 \text{ in.}$$

Natural Frequency for Mode A :

$$f_A := 0.18 \cdot \sqrt{\frac{386}{\Delta_b}} \quad f_A = 6.93 \text{ Hz}$$

Mode B

Uniform Distributed Loading : $w_g := \left(\frac{L_{mb} + L_{ab}}{2} \right) \cdot \left(\frac{w_j}{b_{eff_b}} \right) + w_{mg} \quad w_g = 3271 \text{ lb./ft}$

Beam/Joist Deflection : Secondary Framing (fixed-ended boundary conditions)

$$\delta_b := \frac{w_j \cdot L_{mb}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{b_comp}} \quad \delta_b = 0.052 \text{ in.}$$

Girder Deflection : Primary Framing (pin-pin boundary conditions)

$$\delta_g := \frac{5 \cdot w_g \cdot L_{mg}^4 \cdot 1728}{384 E_s \cdot 1000 \cdot I_{g_comp}} \quad \delta_g = 0.219 \text{ in.}$$

To account for the stiffening effect offered by the adjacent span, the above deflection needs to be modified. Therefore, Equation 3.7 from the AISC publication *Floor Vibrations Due to Human Activity* (Murray, et al. 1997) is used to reduce the deflection of the floor for three continuous spans

Main Girder Properties

$$k_m := \frac{I_{g_comp}}{L_{mg} \cdot 12} \quad k_m = 61.92 \text{ in.}^3$$

Appendix A.2.4
SCI Frequency Calculations
Hot-Rolled Girders with Castelled Beams (Cont.)

AMAT Building 34
File # Boice7
Bay “B”

Adjacent Girder Properties

$$k_s := \frac{I_{g_comp}}{L_{ag} \cdot 12} \quad k_s = 61.92 \text{ in.}^3$$

$$\Delta_g := \left[\frac{0.6 + 2 \cdot \frac{k_m}{k_s} \cdot \left(1 + 1.2 \cdot \frac{L_{ag}^2}{L_{mg}^2} \right)}{3 + 2 \cdot \frac{k_m}{k_s}} \right] \cdot \delta_g \quad \Delta_g = 0.219$$

Natural Frequency for Mode B :

$$f_B := 0.18 \sqrt{\frac{386}{\delta_b + \Delta_g}} \quad f_B = 6.80 \text{ Hz}$$

$f_A > f_B$, therefore, Mode B controls the motion of the Floor and the corresponding Natural Frequency is 6.80 Hz.

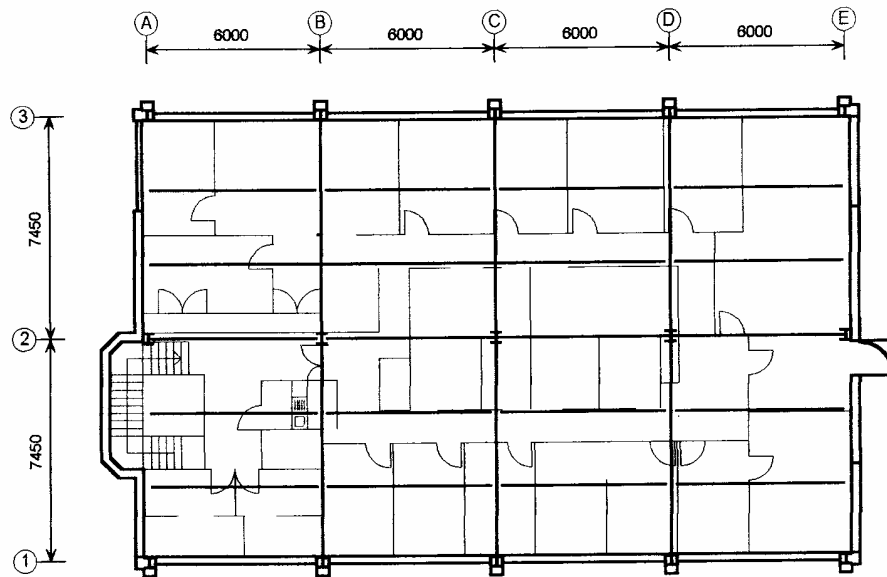
Appendix A.2 SCI Frequency Calculations

A.2.5 BRITISH FLOOR SYSTEM

SCI Headquarters

File # BR3

Bay 1



Main beam sizes:

7.45 m primary beam: 686 × 152 Castellated UB 60

6.00 m secondary beam: 305 × 127 UB 42

First Floor

Floor structure: 130 mm deep normal weight concrete slab cast on top of 1.2 mm thick Super Holorib deck. Slabs supported by 6.0 m span secondary beams at 2.48 m cross-centres which, in turn, are supported by 7.45 m span castellated primary beams in the orthogonal direction on Grid-lines B to D.

LOADING

120 mm deep concrete slab	=	2.79 kN/m ²
Decking	=	0.17 kN/m ²
Ceiling and services	=	0.50 kN/m ²
Raised floor	=	0.30 kN/m ²
Ducting	=	0.10 kN/m ²
10% Imposed	=	0.35 kN/m ²
		4.21 kN/m ²

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

SCI Headquarters

File # BR3

Bay 1

COMPOSITE SLAB PROPERTIES

Data from deck manufacturer -

Deck neutral axis position: 17.28 mm
Deck area: 2124 mm²/m
Deck second moment of area: 8635000 mm⁴/m
Height of re-entrant ribs: 51 mm

Concrete volume for 130 mm deep slab = 0.121 m³/m

∴ Effective slab thickness = 121 mm

For dynamic behaviour, take gross uncracked inertia. Also, for normal weight concrete, take $E_c = 38 \text{ kN/mm}^2$

∴ Modular ratio $\alpha = 205 / 38 = 5.39$

Per metre width:

Section	Area cm ²	y cm	Area × y cm ³
Concrete Deck	$12.1 \times 100 / \alpha = 224.49$	$12.1 / 2 = 6.05$	1358.16
	= 21.24	13 - 1.728 = 11.272	239.42
	<u>245.73</u>		<u>1597.58</u>

∴ Position of composite slab ENA = 1597.58 / 245.73 = 6.50 cm

Section	Distance from ENA cm	Area × Distance ² cm ⁴	I Local cm ⁴
Concrete Deck	$6.50 - 6.05 = 0.45$	45.46	$100 \times 12.1^3 / 12 \alpha = 2738.96$
	$11.272 - 6.50 = 4.77$	483.27	863.5
		<u>528.73</u>	<u>3602.46</u>

∴ Second moment of area of composite slab (per metre width)
= 528.73 + 3602.46 = 4131.19 cm⁴/m (41.31 × 10⁻⁶ m⁴/m)

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

SCI Headquarters

File # BR3

Bay 1

COMPOSITE BEAM PROPERTIES

Secondary beam

Span: 6.00 m (typical) **Average spacing:** 2.48 m

Serial size: 305 × 127 UB 42 **Self weight:** 41.9 kg/m

Depth: 307.2 mm **Area, A:** 53.4 cm²

Second moment of area, I_{beam} : 8196 cm⁴

Span / 4 = 6000 / 4 = 1500 mm (< spacing 2480 mm)

∴ Effective breadth = 1500 mm

Position of ENA

$$= \frac{53.4 \times 10^2 (130 + 307.2 / 2) + ((130 - 51) \times 1500 / \alpha) ((130 - 51) / 2)}{53.4 \times 10^2 + ((130 - 51) \times 1500 / \alpha)}$$

= 87.2 mm

Gross second moment of area:

$$= [8196 \times 10^4 + 53.4 \times 10^2 (130 + 307.2 / 2 - 87.2)^2 + 1500 \times (130 - 51)^3 / (12\alpha) + 1500 \times (130 - 51) / \alpha (87.2 - (130 - 51) / 2)^2] / 10^4 = 34939.63 \text{ cm}^4$$

Primary beam

Span: 7.45 m (typical) **Average spacing:** 6.0 m

Serial size: 686 × 152 Castellated UB 60 **Self weight:** 59.8 kg/m

Depth: 683.1 mm **Area, A:** 57.7 cm²

Second moment of area, I_{beam} : 59230 cm⁴

Span / 4 = 7450 / 4 = 1862.5 mm (< spacing 6000 mm)

∴ Effective breadth = 1862.5 mm

Position of ENA

$$= \frac{57.7 \times 10^2 (130 + 683.1 / 2) + (121 \times 1862.5 / \alpha) \times 121 / 2}{57.7 \times 10^2 + (121 \times 1862.5 / \alpha)}$$

= 110.35 mm

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

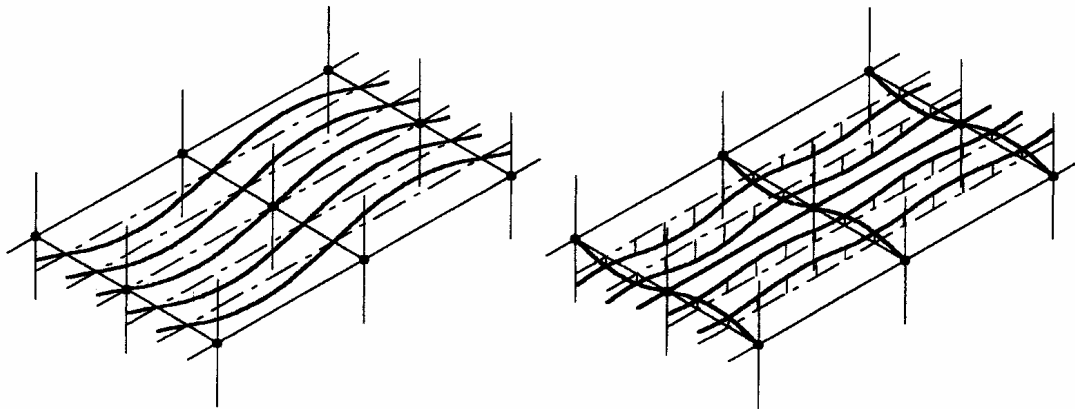
SCI Headquarters
File # BR3
Bay 1

Gross second moment of area:

$$= \left[59230 \times 10^4 + 57.7 \times 10^2 (130 + 683.1 / 2 - 110.35)^2 + 1862.5 \times 121^3 / (12\alpha) + 1862.5 \times 121 / \alpha (110.35 - 121 / 2)^2 \right] / 10^4 = 150000 \text{cm}^4$$

MODE SHAPES & BEAM BOUNDARY CONDITIONS

The fundamental mode is that which gives the lowest natural frequency (i.e., the highest deflexion)



Mode A Secondary beam mode

Mode B Primary beam mode

For mode A, the primary beams form nodal lines (i.e. they have zero deflexion) about which, the secondary beams vibrate as simply-supported members. In this case, the slab flexibility is affected by the approximately equal deflexions of the supports. As a result of this, the slab frequency is assessed on the basis that fixed-ended boundary conditions exist.

For mode B, the primary beams vibrate about the columns as simply-supported members. Using a similar reasoning as above, due to the equal deflexions at their supports, the secondary beams (as well as the slab) are assessed on the basis that fixed-ended boundary conditions exist.

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

SCI Headquarters
File # BR3
Bay 1

NATURAL FREQUENCY

Mode A

i) Slab (fixed-ended)

$$w = 4.21 \times 2.48 = 10.44 \text{ kN/m}$$

$$\delta_s = \frac{wL^3}{384EI} = \frac{10.44 \times 2480^3}{384 \times 205 \times 4131.19 \times 10^4} = 0.05 \text{ mm}$$

ii) Secondary beam (simple-supports)

$$W = 6 (10.44 + 41.9 \times 9.81/10^3) = 65.11 \text{ kN}$$

$$\delta_{sec} = \frac{5wL^3}{384EI} = \frac{5 \times 65.11 \times 6000^3}{384 \times 205 \times 34939.63 \times 10^4} = 2.56 \text{ mm}$$

$$\text{Total deflexion} = 0.05 + 2.56 = 2.61 \text{ mm}$$

$$\therefore \text{Natural frequency for Mode A, } f_A = \frac{18}{\sqrt{\delta}} = \frac{18}{\sqrt{2.61}} = 11.14 \text{ Hz}$$

Mode B

i) Slab

As above

ii) Secondary beam (fixed-ended)

$$\delta_{sec} = \frac{wL^3}{384EI} = \frac{2.56}{5} = 0.51 \text{ mm}$$

iii) Primary beams (simple-supports)

$$P = 65.11 \text{ kN}$$

$$W = 7.45(59.8 \times 9.81/10^3) = 4.37 \text{ kN}$$

$$\delta_p = \frac{23PL^3}{648EI} + \frac{5WL^3}{384EI} = \frac{7450^3}{205 \times 150000 \times 10^4} \left(\frac{23 \times 65.11}{648} + \frac{5 \times 4.37}{384} \right) = 3.18$$

$$\text{Total deflexion} = 0.05 + 0.51 + 3.18 = 3.74 \text{ mm}$$

$$\therefore \text{Natural frequency for Mode B, } f_B = \frac{18}{\sqrt{\delta}} = \frac{18}{\sqrt{3.74}} = 9.31 \text{ Hz}$$

$\therefore$ Mode B governs and, hence, the fundamental frequency of the floor $f_0 = 9.31 \text{ Hz}$

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

SCI Headquarters
File # BR3
Bay 1

FLOOR RESPONSE (SCI P076)

As $f_0 > 7.0$ Hz, the response is to be assessed according to the 'high frequency floor' recommendations

$$\text{Mass } m = (3 \times 65.11 + 4.37) / (6 \times 7.45) \times 1000 / 9.81 = 455.41 \text{ kg/m}^2$$

$$40 \times 0.13 = 5.2 \text{ m} > 2.48 \text{ m spacing of secondary beams}$$

$$\therefore b_e = 2.48 \text{ m}$$

$$L = \text{secondary beam span} = 6.0 \text{ m}$$

The response factor is therefore:

$$R = \frac{30000}{mb_e L} = \frac{30000}{455.41 \times 2.48 \times 6} = 4.4$$

Parameter	Calculated	Measured	Correction factor
Frequency	9.41	8.38	0.89
Response	4.4	2.8	0.64

FLOOR RESPONSE (SCI AD256)

As the primary beams govern the mode shape, the appropriate floor layout from Table 7.1 in SCI P076 is Case (4).

$$\text{Relative flexibility of primary beam } RF_{\text{main beam}} = 3.18/3.74 = 0.85$$

From Table 7.1, as $RF_{\text{main beam}} > 0.6$:

$$L^* = 3.8 \left(\frac{EI_b}{mbf_0^2} \right)^{1/4} = 3.8 \left(\frac{205 \times 34939.63 \times 10}{455.41 \times 2.483 \times 9.31^2} \right)^{1/4} = 19.76 \text{ m}$$

Due to the presence of the stair-well, $L_{\text{max}} = 3 \times 6 = 18 \text{ m} > W$

$$\text{As } L_{\text{max}} < L^*, L_{\text{eff}} = L_{\text{max}} = 18 \text{ m}$$

$$W_1 = 7.45 \text{ m} \quad W_2 = 7.45 \text{ m}$$

From Table 7.1, as $W_2 = W_1$:

$$S = 2W_1 = 14.9 \text{ m}$$

Appendix A.2.5 SCI Frequency Calculations British Floor System (Cont.)

SCI HeadQuarters

File # BR3

Bay 1

Finally, as the floor has partitions orientated perpendicular to the main vibrating elements, the damping $\zeta = 4.5\%$

The response factor is therefore:

$$a_{peak} = \frac{2\alpha_n P}{mSL_{eff}\zeta} = \frac{2 \times 0.1 \times 76 \times 9.81}{455.41 \times 18 \times 14.9 \times 0.045} = 0.027 \text{ m/s}^2$$

$$R = \frac{0.027 \times 8}{\sqrt{2} \times 5 \times 10^{-3} \times 9.31} = 3.3$$

<i>Parameter</i>	<i>Calculated</i>	<i>Measured</i>	<i>Correction factor</i>
<i>Frequency</i>	<i>9.41</i>	<i>8.38</i>	<i>0.89</i>
<i>Damping</i>	<i>4.50%</i>	<i>4.68%</i>	<i>1.04</i>
<i>Response</i>	<i>3.3</i>	<i>2.8</i>	<i>0.85</i>

APPENDIX A.3 SCI ACCELERATION CALCULATIONS

Appendix A.3 SCI Acceleration Calculations

A.3.1 HOT-ROLLED SHAPES

File # 596
Bay Typ. Newport V

$$\begin{aligned} f_c &= 3.50 \text{ ksi} & \text{Total } t_{\text{conc.}} &= 5.25 \text{ in.} \\ w_{\text{conc.}} &= 115 \text{ pcf} & \text{Deck Hgt. } d_h &= 2.00 \text{ in.} \\ P &= 167.6 \text{ lbs} & d_e &= 4.25 \text{ in.} \\ \zeta &= 0.025 \\ \alpha_n &= 0.1 \end{aligned}$$

Mass

$$\begin{aligned} \text{Girder (primary) tributary width} &= 41.75 \text{ ft} & M_{\text{girder}} &= 2.012 \text{ psf} \\ \text{Beam (secondary) spacing} &= 10.28 \text{ ft} & M_{\text{beam}} &= 4.281 \text{ psf} \\ w_{\text{girder}} &= 84.00 \text{ plf} & M_{\text{slab}} &= 41.7 \text{ psf} \\ w_{\text{beam}} &= 44.00 \text{ plf} & M_{\text{D.L.}} &= 16.5 \text{ psf} \\ & & M_{\text{L.L.}} &= 11.0 \text{ psf} \end{aligned}$$

$$\begin{aligned} m_{\text{tot}} &= (M_{\text{girder}} + M_{\text{beam}} + M_{\text{slab}} + M_{\text{D.L.}} + M_{\text{L.L.}}) / (32.2 \text{ ft/s}^2) \\ m_{\text{tot}} &= 2.345 \text{ lb*s}^2/\text{ft}^3 \end{aligned}$$

Resonance Build-Up Factor (R_I)

$$\begin{aligned} R_I &= 1 - e^{(-2\pi f_o \zeta T_w)} & T_w &= D/V \\ f_{\text{pred}} = f_o &= 3.84 \text{ Hz} & D &= 270.00 \text{ ft} & f_p &= 1.920 \text{ Hz} \\ f_q &= 0.00 \text{ Hz} \\ f_b &= 3.84 \text{ Hz} & V &= 1.67f_p^2 - 4.83f_p + 4.50 & 1.7 \text{ Hz} &\leq f_p \leq 2.4 \text{ Hz} \\ & & V &= 4.536 \text{ ft/s} & T_w &= D/V = 59.52 \text{ s} \\ R_I &= 1 - e^{(-2\pi f_o \zeta T_w)} = 1.00 \end{aligned}$$

Find L_{eff} and S

$$\begin{aligned} \text{Secondary Beam Controls (Mode A)} & & \delta_g &= 0.000 \text{ in} \\ \text{Assuming Case 2 Governs} & & \delta_b &= 0.846 \text{ in} \\ L &= 45.83 \text{ ft} & RF_{m.b.} &= \delta_g / (\delta_g + \delta_b) \\ L_{\text{eff}} = 1.7L &= 77.91 \text{ ft} & RF_{m.b.} &= 0.0000 < 0.2 \\ E = w_{\text{conc.}}^{1.5} (f_c)^{1/2} &= 2307 \text{ ksi} & \tau_I &= ((12 \text{ in.} * d_e^3) / 12) / 12 \text{ in.} = 6.397 \text{ in.}^4/\text{in.} \\ S^* = 4.5 * (1.35EI_1 / (mf_o^2))^{1/4} &= 66.62 \text{ ft} \\ S = S^* &= 66.62 \text{ ft} & & \text{<----- controls} \\ S^* \leq W &= 270.00 \text{ ft} \\ a_{\text{peak}} &= [(2\alpha_n P) / (m_{\text{tot}} L_{\text{eff}} S \zeta)] * R_I \end{aligned}$$

SCI AD256
$a_{\text{peak}}/g = 0.34 \%$

Design Guide 11
$a_{\text{peak}}/g = 0.46\%$

Check Acceptability

$$a_{\text{rms}}/g < (0.005R/g) \quad a_{\text{rms}} = a_{\text{peak}}/2^{0.5}$$

$a_{\text{rms}}/g = 0.24 \%$

Special Office -- R =	4	0.20 %	No good for this case
General Office -- R =	8	0.41 %	Good for this case
Busy Office -- R =	12	0.61 %	Good for this case

Appendix A.3 SCI Acceleration Calculations

A.3.2 HOT-ROLLED GIRDERS WITH JOISTS

File # 563
Bay Typ. **Melroe Corp.**

$f_c =$	3.00 ksi	Total $t_{conc.} =$	2.75 in.
$w_{conc.} =$	145 pcf	Deck Hgt. $d_h =$	0.63 in.
$P =$	167.6 lbs	$d_e =$	2.44 in.
$\zeta =$	0.03		
$\alpha_n =$	0.1		

Mass

Girder (primary) tributary width =	39.25 ft	$M_{girder} =$	1.936 psf
Beam (secondary) spacing =	2.00 ft	$M_{beam} =$	5.750 psf
$w_{girder} =$	76.00 plf	$M_{slab} =$	30.50 psf
$w_{beam} =$	11.50 plf	$M_{D.L.} =$	7 psf
		$M_{L.L.} =$	11 psf

$$m_{tot} = (M_{girder} + M_{beam} + M_{slab} + M_{D.L.} + M_{L.L.}) / (32.2 \text{ ft/s}^2)$$

$$m_{tot} = 1.745 \text{ lb*s}^2/\text{ft}^3$$

Resonance Build-Up Factor (R_1)

$$R_1 = 1 - e^{(-2\pi f_o \zeta T_w)}$$

$$T_w = D/V$$

$f_{pred} = f_o =$	4.57 Hz	$D =$	210.00 ft	$f_p =$	2.285 Hz
$f_g =$	4.93 Hz				
$f_b =$	12.21 Hz	$V = 1.67f_p^2 - 4.83f_p + 4.50$		$1.7 \text{ Hz} \leq f_p \leq 2.4 \text{ Hz}$	

$$V = 7.162 \text{ ft/s} \quad T_w = D/V = 29.32 \text{ s}$$

$$R_1 = 1 - e^{(-2\pi f_o \zeta T_w)} = 1.00$$

Find L_{eff} and S

Primary Beam Controls (Mode B)		$\delta_g =$	0.514 in
Assuming Case 4 Governs		$\delta_b =$	0.084 in
$W =$	30.00 ft	$RF_{m.b.} =$	$\delta_g/(\delta_g+\delta_b)$
$S = 2W =$	60.00 ft	$RF_{m.b.} =$	0.8595 > 0.6
$E_s =$	29000 ksi	$I_b =$	435.10 in ⁴

$$L^* = 3.8 * (E_s I_b / (m b f_o^2))^{1/4} = 125.83 \text{ ft} \quad L_{max} = 38.38 \text{ ft}$$

$$L^* \leq L_{max}, \text{ therefore } L_{eff} = 38.38 \text{ ft}$$

$$a_{peak} = [(2\alpha_n P) / (m_{tot} L_{eff} S \zeta)] * R_1$$

SCI AD256 $a_{peak}/g = 0.86 \%$
--

Design Guide 11 $a_{peak}/g = 0.74\%$

Check Acceptability

$$a_{rms}/g < (0.005R/g)$$

$$a_{rms} = a_{peak}/2^{0.5}$$

$a_{rms}/g = 0.61 \%$

Special Office -- $R =$	4	0.20 %	No good for this case
General Office -- $R =$	8	0.41 %	No good for this case
Busy Office -- $R =$	12	0.61 %	Good for this case

Appendix A.3 SCI Acceleration Calculations

A.3.3 JOIST GIRDERS WITH JOISTS

File #	506	Garmin International	
Bay	Ext./Int.		
f'_c	3.50 ksi	Total $t_{conc.}$	2.50 in.
$w_{conc.}$	150 pcf	Deck Hgt. d_h	0.63 in.
P	167.6 lbs	d_e	2.19 in.
ζ	0.025		
α_n	0.1		

Mass

Girder (primary) tributary width	=	28.00 ft	M_{girder}	=	0.968 psf
Beam (secondary) spacing	=	2.50 ft	M_{beam}	=	2.880 psf
w_{girder}	=	27.10 plf	M_{slab}	=	28.30 psf
w_{beam}	=	7.20 plf	$M_{D.L.}$	=	4 psf
			$M_{L.L.}$	=	11 psf

$$m_{tot} = (M_{girder} + M_{beam} + M_{slab} + M_{D.L.} + M_{L.L.}) / (32.2 \text{ ft/s}^2)$$

$$m_{tot} = 1.464 \text{ lb*s}^2/\text{ft}^3$$

Resonance Build-Up Factor (R_I)

$$R_I = 1 - e^{(-2\pi f_o \zeta T_w)} \quad T_w = D/V$$

$$f_{pred} = f_o = 3.94 \text{ Hz} \quad D = 345.00 \text{ ft} \quad f_p = 1.970 \text{ Hz}$$

$$f_g = 4.10 \text{ Hz}$$

$$f_b = 13.97 \text{ Hz} \quad V = 1.67 f_p^2 - 4.83 f_p + 4.50 \quad 1.7 \text{ Hz} \leq f_p \leq 2.4 \text{ Hz}$$

$$V = 4.810 \text{ ft/s} \quad T_w = D/V = 71.73 \text{ s}$$

$$R_I = 1 - e^{(-2\pi f_o \zeta T_w)} = 1.00$$

Find L_{eff} and S

Primary Beam Controls (Mode B)	δ_g	=	0.743 in
Assuming Case 4 Governs	δ_b	=	0.064 in
W	35.00 ft	$RF_{m.b.}$	$\delta_g / (\delta_g + \delta_b)$
$S = 2W$	70.00 ft	$RF_{m.b.}$	0.9205 > 0.6
E_s	29000 ksi	I_b	171.76 in ⁴
$L^* = 3.8 * (E_s I_b / (m b f_o^2))^{1/4}$	106.14 ft	L_{max}	28.00 ft
$L^* \leq L_{max}$, therefore	L_{eff}	=	28.00 ft

$$a_{peak} = [(2\alpha_n P) / (m_{tot} L_{eff} S \zeta)] * R_I$$

SCI AD256
$a_{peak}/g = 1.45 \%$

Design Guide 11
$a_{peak}/g = 1.08 \%$

Check Acceptability

$$a_{rms}/g < (0.005R/g) \quad a_{rms} = a_{peak}/2^{0.5}$$

$a_{rms}/g = 1.03 \%$

Special Office -- R	=	4	0.20 %	No good for this case
General Office -- R	=	8	0.41 %	No good for this case
Busy Office -- R	=	12	0.61 %	No good for this case

Appendix A.3 SCI Acceleration Calculations

A.3.4 HOT-ROLLED GIRDERS WITH CASTELLATED BEAMS

File # Boice7 AMAT
Bay B

$f_c =$	4.00 ksi	Total $t_{conc.} =$	7.00 in.
$w_{conc.} =$	145 pcf	Deck Hgt. $d_h =$	2.00 in.
$P =$	167.6 lbs	$d_e =$	6.00 in.
$\zeta =$	0.03		
$\alpha_n =$	0.1		

Mass

Girder (primary) tributary width =	40.00 ft	$M_{girder} =$	4.550 psf
Beam (secondary) spacing =	8.00 ft	$M_{beam} =$	3.725 psf
$w_{girder} =$	182.00 plf	$M_{slab} =$	73.50 psf
$w_{beam} =$	29.80 plf	$M_{D.L.} =$	4 psf
		$M_{L.L.} =$	11 psf

$$m_{tot} = (M_{girder} + M_{beam} + M_{slab} + M_{D.L.} + M_{L.L.}) / (32.2 \text{ ft/s}^2)$$

$$m_{tot} = 3.005 \text{ lb*s}^2/\text{ft}^3$$

Resonance Build-Up Factor (R_I)

$R_I = 1 - e^{(-2\pi f_o \zeta T_w)}$	$T_w = D/V$		
$f_{pred} = f_o =$	6.24 Hz	$D =$	120.00 ft
$f_g =$	6.95 Hz	$f_p =$	2.080 Hz
$f_b =$	14.19 Hz	$V = 1.67f_p^2 - 4.83f_p + 4.50$	$1.7 \text{ Hz} \leq f_p \leq 2.4 \text{ Hz}$
		$V =$	5.508 ft/s
		$T_w = D/V =$	21.79 s

$$R_I = 1 - e^{(-2\pi f_o \zeta T_w)} = 1.00$$

Find L_{eff} and S

Primary Beam Controls (Mode B)	$\delta_g =$	0.259 in
Assuming Case 4 Governs	$\delta_b =$	0.062 in
$W =$	40.00 ft	$RF_{m.b.} = \delta_g / (\delta_g + \delta_b)$
$S = 2W =$	80.00 ft	$RF_{m.b.} = 0.8064 > 0.6$
$E_s =$	29000 ksi	$I_b =$
		4716.67 in ⁴
$L^* = 3.8 * (E_s I_b / (m b f_o^2))^{1/4} =$	120.60 ft	$L_{max} =$
$L^* \leq L_{max}$, therefore	$L_{eff} =$	120.00 ft

$$a_{peak} = [(2\alpha_n P) / (m_{tot} L_{eff} S \zeta)] * R_I$$

SCI AD256
$a_{peak}/g = 0.12 \%$

Design Guide 11
$a_{peak}/g = 0.17\%$

Check Acceptability

$$a_{rms}/g < (0.005R/g)$$

$$a_{rms} = a_{peak}/2^{0.5}$$

$a_{rms}/g = 0.09 \%$

Special Office -- $R =$	4	0.20 %	Good for this case
General Office -- $R =$	8	0.41 %	Good for this case
Busy Office -- $R =$	12	0.61 %	Good for this case

APPENDIX A.4 MURRAY CRITERION CALCULATIONS

Appendix A.4 Murray Criterion Calculations

A.4.1 HOT-ROLLED SHAPES

Newport V
File # 596
Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

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537 Wisteria Drive
Radford, VA 24141

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 1

Span= 30.833 ft	Beam	Right Girder:	Defaults
Left Girder:	W21x44	W27x84	Table: FLOORVIB.VDT
Wall	3 Spc @ 123.33		Estimated Damping:
			Bare Floor: 1 %
Left		Right	Ceiling: %
Bm. Span:	Bm Span= 45.83 ft	Bm. Span:	Mech.: %
0.0 ft		37.667 ft	Partitions: %
			Total: 1 %
Concrete: dc: 5-1/4 in.(total depth)			Office/Residential Activity
fc': 3.5 Ksi			Design Criterion: Murray
wt.: 115 pcf			System Evaluation:
Deck Height: 2"			DOES NOT SATISFY CRITERION
Loading: Dead: 4 psf			Est. damping = 1.0% < Req'd 3.4%
Collateral: 12.5 psf			<Press [F1] for Advice>
(v) Live: 8 psf			

VIBRATION ANALYSIS:

Evaluation Criterion: Murray
Environment: Office/Residential
Reference: Murray, T.M., "Building Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1991,
pages 102-109.

Required damping, D (%) = 35 Ao f + 2.5

	Section	w, plf	Itr, in4	f, Hz.	Ao, in.	Dreq'd, %
Beam	W21x44	724.4	2669.5	3.67	0.0068	3.4
Left Girder	Wall	1615.1	0.0	10000.	0.0	0.0
Right Girder	W27x84	3027.0	7389.0	6.57	0.0027	3.1
Bay (Using smaller girder frequency)				3.20	0.0081	3.4

Required damping is 0.0 in the above table because the natural frequency is greater than 10 Hz.

Evaluation:

The Beam controls
The Bay controls

-- Continued --

Appendix A.4.1

Murray Criterion Calculations

Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 2

Required damping = 3.4 % > 1 % Actual damping
The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 41.7 psf
Dead loads = 4.0 psf
Collateral loads = 12.5 psf
Live loads = 8.0 psf
Actual beam and girder weights

Tributary width for left girder = $45.83/2 = 22.92$ ft.
Tributary width for right girder = $45.83/2 + 37.67/2 = 41.75$ ft.

ESTIMATED DAMPING:

Bare floor:	1.0 %
Ceiling:	0.0 %
Mechanical:	0.0 %
Partition:	0.0 %
Total:	1.0 %

CONCRETE/SLAB DATA:

Concrete	dc= 5.25 in.	fc'= 3.5 Ksi
	wt= 115 pcf	Ec= 2307 Ksi

Modular ratio, n= 12.57

Deck height: 2 in.
Effective concrete thickness in deck: 1 in.

BEAM CALCULATIONS:

Beam section: W21x44 d= 20.660 in. A= 13.00 in.² Ix= 843 in.⁴

Spacing: S = 123.33 in.
Span: Lb = 45.83 ft.
Uniform load: wb = $(41.7 + 4.0 + 12.5 + 8.0) \times 123.33/12 + 44.0$
= 724.4 plf

Transformed moment of inertia:
Effective concrete width = min(0.4 Lb, S) = 123.330 in.
Effective concrete depth = 3.250 in.
Transformed concrete width = 9.811 in.
Transformed concrete area = 31.886 in.²
Distance to neutral axis = 9.913 in. (Above beam c. g.)
Transformed moment of inertia = 2669.5 in.⁴

-- Continued --

Appendix A.4.1 Murray Criterion Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 3

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{w_b L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 2669.5}{(724.4 / 12) \times (45.83 \times 12)^4} \right]^{0.5} = 3.67 \text{ Hz.}$$

Tee-Beam Amplitude:

$$\alpha = 0.1 \pi f_b = 1.153$$

$$\text{since } t_o = [1/(\pi f_b)] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 3.67)] \times 0.86$$

$$= 0.0743 \text{ sec.} > 0.05 \text{ sec.}$$

$$A_{ot} = \frac{246 L^3}{\text{Es I}_b} \frac{1}{2 \pi f_b} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0147 \text{ in.}$$

Number of Effective Tee-Beams:

$$N_{eff} = 2.97 - S/(17.3 d_e) + L^4/(1.35 \text{ Es Itr})$$

$$= 2.97 - 123.33/(17.3 \times 4.25)$$

$$+ (45.83 \times 12)^4/(1.35 \times 29 \times 10^6 \times 2669.49)$$

$$= 2.17$$

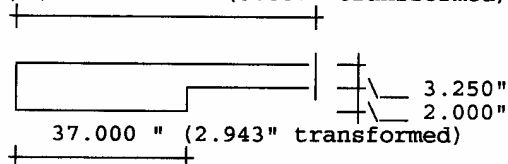
$$\text{Beam Amplitude, } A_{ob} = A_{ot}/N_{eff} = 0.0068 \text{ in.}$$

LEFT GIRDER CALCULATIONS:

Tributary width = 22.92 ft.
Span: Lg = 30.83 ft.
Equivalent uniform load: wg = 22.92 x (724.4/10.28) + 0.0
= 1615.1 plf

Transformed moment of inertia:

$$\min(0.2 L_g, L_b/2) = 73.999 \text{ " (5.887" transformed)}$$



Effective concrete width = 73.999 in. and 37.000 in.
Transformed concrete width = 5.887 in. and 2.943 in.
Transformed concrete area = 19.132 in.² and 5.887 in.²
Distance to neutral axis = 0.0 in. (Above girder c. g.)

-- Continued --

Appendix A.4.1

Murray Criterion Calculations

Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 4

Transformed moment of inertia = 0.0 in.4

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{\text{wg L}^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es x } 0.0}{(1615.1/12) \times (30.83 \times 12)^4} \right]^{0.5} = 10000.00 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi \text{ fg} = 0.0$$

$$\text{since } t_o = [1/(\pi \text{ fg})] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 10000.00)] \times 0.0$$

$$= 0.0 \text{ sec.} \leq 0.05 \text{ sec.}$$

$$\text{Aog} = [246 \text{ L}^3 / (\text{Es Ig})] (0.1 - t_o) = 0.0 \text{ in.}$$

RIGHT GIRDER CALCULATIONS:

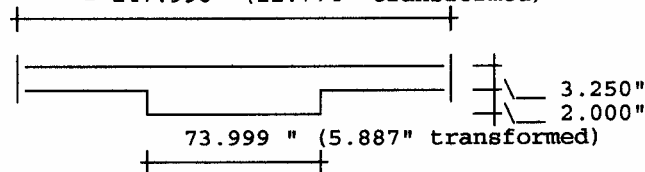
Girder section: W27x84 d= 26.710 in. A= 24.80 in.2 Ix= 2850 in.4

Tributary width = 41.75 ft.
Span: Lg = 30.83 ft.
Equivalent uniform load: wg = 41.75 x (724.4/10.28) + 84.0
= 3027.0 plf

Transformed moment of inertia:

$$\min(0.2 \text{ Lg}, 45.83 \times 12/2) + \min(0.2 \text{ Lg}, 37.67 \times 12/2)$$

$$= 147.998" (11.774" \text{ transformed})$$



Effective concrete width = 147.998 in. and 73.999 in.
Transformed concrete width = 11.774 in. and 5.887 in.
Transformed concrete area = 38.264 in.2 and 11.774 in.2
Distance to neutral axis = 10.940 in. (Above girder c. g.)
Transformed moment of inertia = 7389.0 in.4

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{\text{wg L}^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es x } 7389.0}{\text{wg L}^4} \right]^{0.5} = 6.57 \text{ Hz.}$$

-- Continued --

Appendix A.4.1 Murray Criterion Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 5

$$L = (3027.0/12) \times (30.83 \times 12)^4$$

Girder Amplitude:

$$\alpha = 0.1 \pi f_g = 2.064$$

$$\begin{aligned} \text{since } t_o &= [1/(\pi f_g)] \tan^{-1}(\alpha) \\ &= [1/(\pi \times 6.57)] \times 1.12 \\ &= 0.0542 \text{ sec.} > 0.05 \text{ sec.} \end{aligned}$$

$$A_{og} = \frac{246 L^3}{E_s I_g} \frac{1}{2 \pi f_g} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0027 \text{ in.}$$

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\text{Frequency: } \frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{3.67^2} + \frac{1}{6.57^2}$$

$$f_s = 3.20 \text{ Hz.}$$

$$\text{Amplitude: } A_{os} = A_{ob} + A_{og}/2 = 0.0068 + 0.0027/2 = 0.0081 \text{ in.}$$

Warning:

The floor system does not satisfy the Murray criterion.
Possible solutions are:

- Increase the concrete thickness, which will decrease the natural frequency but will not significantly change the initial amplitude, A_o .
- Change the beam spacing.
- Use normal weight concrete, which will decrease the natural frequency but will not significantly change the initial amplitude, A_o .

Generally, increasing the member stiffness (moment of inertia) will not improve the evaluation.

-- End --

Appendix A.4 Murray Criterion Calculations

A.4.2 HOT-ROLLED GIRDERS WITH JOISTS

Melroe Corporation

File # 563

Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

Licensed to: STRUCTURAL ENGINEERS, INC.
537 Wisteria Drive
Radford, VA 24141

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 1

Span= 30 ft			Defaults Table: FLOORVIB.VDT	
Left Girder: W24x76	Joist 24K8 15Spc @ 24"	Right Girder: W24x76	Estimated Damping:	
Left Bm. Span: 40.125 ft	Bm Span= 38.375 ft	Right Bm. Span: 40.125 ft	Bare Floor:	1 %
			Ceiling:	%
			Mech.:	%
			Partitions:	%
			Total:	1 %
Concrete: dc: 2-3/4 in.(total depth)	Office/Residential Activity			
fc': 3 Ksi	Design Criterion: Murray			
wt.: 145 pcf	System Evaluation:			
Deck Height: 5/8"	DOES NOT SATISFY CRITERION			
Loading: Dead: 7 psf	Est. damping = 1.0% < Req'd 4.4%			
Collateral: psf	<Press [F1] for Advice>			
(V) Live: 11 psf				

VIBRATION ANALYSIS:

Evaluation Criterion: Murray
Environment: Office/Residential
Reference: Murray, T.M., "Building Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1991,
pages 102-109.

Required damping, D (%) = $35 A_o f + 2.5$

	Section	w, plf	Itr, in4	f, Hz.	Ao, in.	Dreq'd, %
Joist	24K8	108.5	451.4	5.56	0.0099	4.4
Left Girder	W24x76	2204.4	5664.4	7.12	0.0034	3.3
Right Girder	W24x76	2204.4	5664.4	7.12	0.0034	3.3
Bay (Using smaller girder frequency)				4.38	0.0116	4.3

Evaluation:

The Joist controls
Required damping = 4.4 % > 1 % Actual damping
The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

-- Continued --

Appendix A.4.2
Murray Criterion Calculations
Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 2

Slab + 1.0 psf Deck = 30.5 psf
Dead loads = 7.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for girder = $40.13/2 + 38.38/2 = 39.25$ ft.

ESTIMATED DAMPING:

Bare floor:	1.0 %
Ceiling:	0.0 %
Mechanical:	0.0 %
Partition:	0.0 %
Total:	1.0 %

CONCRETE/SLAB DATA:

Concrete	dc= 2.75 in.	fc' = 3 Ksi
	wt= 145 pcf	Ec= 3024 Ksi

Modular ratio, $n = 9.59$

Deck height: 0.625 in.

Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 24K8 (11.5 plf)
 $d = 24.000$ in. $A = 2.157$ in.² $I_x = 282$ in.⁴ $y_c = 10.72$ in.

Spacing: $S = 24$ in.

Span: $L_b = 38.38$ ft.

Uniform load: $w_b = (30.5 + 7.0 + 0.0 + 11.0) \times 24.00/12 + 11.5$
 $= 108.5$ plf

Transformed moment of inertia:

Effective concrete width = $\min(0.4 L_b, S) = 24.000$ in.

Effective concrete depth = 2.125 in.

Transformed concrete width = 2.503 in.

Transformed concrete area = 5.318 in.²

Distance to neutral axis = 9.228 in. (Above beam c. g.)

Transformed moment of inertia = 451.4 in.⁴

$6 \leq L_j/D = 19.19 \leq 24$

$Cr = 0.9 [1 - e^{(-0.28 L_j/D)}]^{2.8} = 0.89$

$\tau = (1/Cr) - 1 = 0.126$

Effective moment of inertia = $1/[\tau/I_{chords} + 1/I_{comp}] = 375.7$ in.⁴

-- Continued --

Appendix A.4.2 Murray Criterion Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 3

$$\begin{aligned} \text{Frequency} &= 1.57 \times \left[\frac{386 \text{ Es Itr}}{w_b L^4} \right]^{0.5} \\ &= 1.57 \times \left[\frac{386 \text{ Es} \times 451.4}{(108.5 / 12) \times (38.38 \times 12)^4} \right]^{0.5} = 5.56 \text{ Hz.} \end{aligned}$$

Tee-Beam Amplitude:

$$\begin{aligned} \alpha &= 0.1 \pi f_b = 1.747 \\ \text{since } t_o &= [1/(\pi f_b)] \tan^{-1}(\alpha) \\ &= [1/(\pi \times 5.56)] \times 1.05 \\ &= 0.0602 \text{ sec.} > 0.05 \text{ sec.} \end{aligned}$$

$$A_{ot} = \frac{246 L^3}{\text{Es I}_b} \frac{1}{2 \pi f_b} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0736 \text{ in.}$$

Number of Effective Tee-Beams:

$$\begin{aligned} D_x &= E_c d_e^3 / 12 = 3024 \times 2.44^3 / 12 = 3661 \text{ K-in.} \\ D_y &= \text{Es Itr} / S = 29000 \times 451.4 / 24.00 = 545442 \text{ K-in.} \\ \epsilon &= (D_x/D_y)^{0.25} = 0.29 \\ X_o &= 1.06 \epsilon L_b = 1.06 \times 0.29 \times 460.5 = 139.71 \text{ in.} \\ X &= 24", 48", 72", 96", 120" \end{aligned}$$

$$\begin{aligned} N_{eff} &= 1.0 + 2 \sum \cos(\pi x / (2 x_o)) \\ &= 7.41 \end{aligned}$$

$$\text{Beam Amplitude, } A_{ob} = A_{ot}/N_{eff} = 0.0099 \text{ in.}$$

LEFT GIRDER CALCULATIONS:

$$\text{Girder section: } W24 \times 76 \quad d = 23.920 \text{ in.} \quad A = 22.40 \text{ in.}^2 \quad I_x = 2100 \text{ in.}^4$$

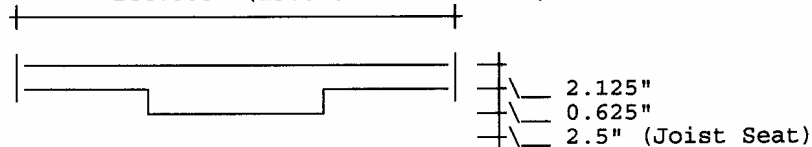
$$\text{Tributary width} = 39.25 \text{ ft.}$$

$$\text{Span: } L_g = 30.00 \text{ ft.}$$

$$\begin{aligned} \text{Equivalent uniform load: } w_g &= 39.25 \times (108.5/2.00) + 76.0 \\ &= 2204.4 \text{ plf} \end{aligned}$$

Transformed moment of inertia:

$$\begin{aligned} &\min(0.2 L_g, 40.13 \times 12/2) + \min(0.2 L_g, 38.38 \times 12/2) \\ &= 144.000" \text{ (15.016" transformed)} \end{aligned}$$



-- Continued --

Appendix A.4.2

Murray Criterion Calculations

Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 4

72.000 " (7.508" transformed)
+-----+

Effective concrete width = 144.000 in. and 72.000 in.
Transformed concrete width = 15.016 in. and 7.508 in.
Transformed concrete area = 31.908 in.² and 4.692 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 9.908 in. (Above girder c. g.)
Transformed moment of inertia = 5664.4 in.⁴

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{\text{wg L}^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 5664.4}{(2204.4/12) \times (30.00 \times 12)^4} \right]^{0.5} = 7.12 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi \text{ fg} = 2.237$$

$$\text{since } t_o = [1/(\pi \text{ fg})] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 7.12)] \times 1.15$$

$$= 0.0514 \text{ sec.} > 0.05 \text{ sec.}$$

$$A_{og} = \frac{246 \text{ L}^3}{\text{Es I}_g} \frac{1}{2 \pi \text{ fg}} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0034 \text{ in.}$$

RIGHT GIRDER CALCULATIONS:

Girder section: W24x76 d= 23.920 in. A= 22.40 in.² Ix= 2100 in.⁴

Same as Left girder.

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\text{Frequency: } \frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{5.56^2} + \frac{1}{7.12^2}$$

$$f_s = 4.38 \text{ Hz.}$$

$$\text{Amplitude: } A_{os} = A_{ob} + A_{og}/2 = 0.0099 + 0.0034/2 = 0.0116 \text{ in.}$$

Warning:

The floor system does not satisfy the Murray criterion.
Possible solutions are:

-- Continued --

Appendix A.4.2
Murray Criterion Calculations
Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 5

-
- Increase the concrete thickness, which will decrease the natural frequency but will not significantly change the initial amplitude, A_0 .
 - Change the beam spacing.

Generally, increasing the member stiffness (moment of inertia) will not improve the evaluation.

-- End --

Appendix A.4 Murray Criterion Calculations

A.4.3 JOIST GIRDERS WITH JOISTS

Garmin International

File # 506

Typical Exterior Bay

Knowledge Base FLOORVIB2 Version 1.1B2

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537 Wisteria Drive
Radford, VA 24141

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 1

Span= 35 ft		Left Girder: Wall		Joist 18K4 14Spc @ 30"		Right Girder: 36G14N4.05		Defaults Table: FLOORVIB.VDT	
Left Bm. Span: 0.0 ft		Bm Span= 28 ft		Right Bm. Span: 28 ft		Estimated Damping:		Bare Floor: 1 %	
								Ceiling: %	
								Mech.: %	
								Partitions: %	
								Total: 1 %	
Concrete: dc: 2-1/2 in.(total depth) fc': 3.5 Ksi wt.: 150 pcf				Office/Residential Activity Design Criterion: Murray System Evaluation: DOES NOT SATISFY CRITERION Est. damping = 1.0% < Req'd 5.8% <Press [F1] for Advice>					
Deck Height: 5/8"									
Loading: Dead: 4 psf									
Collateral: psf									
(V) Live: 11 psf									

VIBRATION ANALYSIS:

Evaluation Criterion: Murray
Environment: Office/Residential
Reference: Murray, T.M., "Building Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1991,
pages 102-109.

Required damping, D (%) = 35 Ao f + 2.5

	Section	w, plf	Itr, in4	f, Hz.	Ao, in.	Dreq'd, %
Joist	18K4	115.4	183.9	6.46	0.0145	5.8
Left Girder	Wall	708.5	0.0	10000.	0.0	0.0
Right Girder	36G14N4.	1320.8	4161.1	5.79	0.0063	3.8
Bay (Using smaller girder frequency)				4.31	0.0176	5.2

Required damping is 0.0 in the above table because the natural frequency is greater than 10 Hz.

Evaluation:

The Joist controls
Required damping = 5.8 % > 1 % Actual damping

-- Continued --

Appendix A.4.3
Murray Criterion Calculations
Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 2

The system DOES NOT SATISFY THE CRITERION.

LOADING DATA:

Slab + 1.0 psf Deck = 28.3 psf
Dead loads = 4.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for left girder = $28.00/2 = 14.00$ ft.
Tributary width for right girder = $2 \times (28.00/2) = 28.00$ ft.

ESTIMATED DAMPING:

Bare floor:	1.0 %
Ceiling:	0.0 %
Mechanical:	0.0 %
Partition:	0.0 %
Total:	1.0 %

CONCRETE/SLAB DATA:

Concrete	dc= 2.50 in.	fc'= 3.5 Ksi
	wt= 150 pcf	Ec= 3437 Ksi

Modular ratio, $n = 8.44$

Deck height: 0.625 in.
Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 18K4 (7.2 plf)
d= 18.000 in. A= 1.431 in.² Ix= 102 in.⁴ yc= 8.06 in.

Spacing: S = 30 in.
Span: Lb = 28.00 ft.
Uniform load: wb = $(28.3 + 4.0 + 0.0 + 11.0) \times 30.00/12 + 7.2$
= 115.4 plf

Transformed moment of inertia:
Effective concrete width = $\min(0.4 L_b, S) = 30.000$ in.
Effective concrete depth = 1.875 in.
Transformed concrete width = 3.556 in.
Transformed concrete area = 6.667 in.²
Distance to neutral axis = 8.134 in. (Above beam c. g.)
Transformed moment of inertia = 183.9 in.⁴
 $6 \leq L_j/D = 18.67 \leq 24$

-- Continued --

Appendix A.4.3 Murray Criterion Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 3

$$\begin{aligned} Cr &= 0.9 [1 - e^{(-0.28 L_j/D)}]^{2.8} = 0.89 \\ \tau &= (1/Cr) - 1 = 0.128 \\ \text{Effective moment of inertia} &= 1/[\tau/I_{chords} + 1/I_{comp}] = 149.4 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} \text{Frequency} &= 1.57 \times \left[\frac{386 E_s I_{tr}}{w_b L^4} \right]^{0.5} \\ &= 1.57 \times \left[\frac{386 E_s \times 183.9}{(115.4/12) \times (28.00 \times 12)^4} \right]^{0.5} = 6.46 \text{ Hz.} \end{aligned}$$

Tee-Beam Amplitude:

$$\begin{aligned} \alpha &= 0.1 \pi f_b = 2.029 \\ \text{since } t_o &= [1/(\pi f_b)] \tan^{-1}(\alpha) \\ &= [1/(\pi \times 6.46)] \times 1.11 \\ &= 0.0548 \text{ sec.} > 0.05 \text{ sec.} \end{aligned}$$

$$A_{ot} = \frac{246 L^3}{E_s I_b} \frac{1}{2 \pi f_b} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0791 \text{ in.}$$

Number of Effective Tee-Beams:

$$\begin{aligned} D_x &= E_c d_e^3 / 12 = 3437 \times 2.19^3 / 12 = 3008 \text{ K-in.} \\ D_y &= E_s I_{tr} / S = 29000 \times 183.9 / 30.00 = 177770 \text{ K-in.} \\ \epsilon &= (D_x/D_y)^{0.25} = 0.36 \\ X_o &= 1.06 \epsilon L_b = 1.06 \times 0.36 \times 336 = 128.46 \text{ in.} \\ X &= 30", 60", 90", 120" \end{aligned}$$

$$\begin{aligned} N_{eff} &= 1.0 + 2 \sum \cos(\pi x / (2 x_o)) \\ &= 5.47 \end{aligned}$$

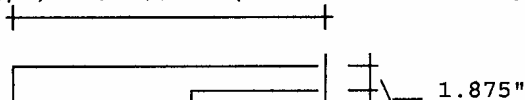
$$\text{Beam Amplitude, } A_{ob} = A_{ot}/N_{eff} = 0.0145 \text{ in.}$$

LEFT GIRDER CALCULATIONS:

$$\begin{aligned} \text{Tributary width} &= 14.00 \text{ ft.} \\ \text{Span: } L_g &= 35.00 \text{ ft.} \\ \text{Equivalent uniform load: } w_g &= 14.00 \times (115.4/2.50) + 62.0 \\ &= 708.5 \text{ plf} \end{aligned}$$

Transformed moment of inertia:

$$\min(0.2 L_g, L_b/2) = 84.000 \text{ " (9.955" transformed)}$$



-- Continued --

Appendix A.4.3 Murray Criterion Calculations Joist Girders with Joists (Cont.)

Garmin International File # 506 Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 4



Effective concrete width = 84.000 in. and 42.000 in.
Transformed concrete width = 9.955 in. and 4.978 in.
Transformed concrete area = 18.666 in.² and 3.111 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 0.0 in. (Above girder c. g.)
Transformed moment of inertia = 0.0 in.⁴

$$\text{Frequency} = 1.57 \times \left[\frac{386 E_s I_{tr}}{w_g L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 E_s \times 0.0}{(708.5 / 12) \times (35.00 \times 12)^4} \right]^{0.5} = 10000.00 \text{ Hz.}$$

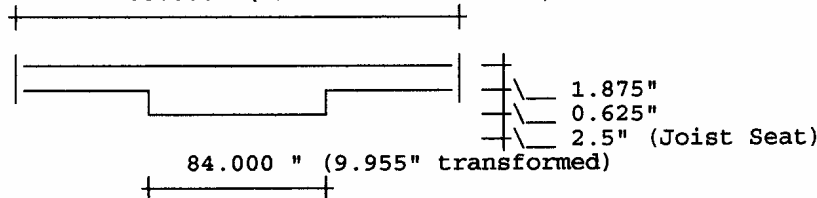
Girder Amplitude:
 $\alpha = 0.1 \pi f_g = 0.0$
since $t_o = [1/(\pi f_g)] \tan^{-1}(\alpha)$
= $[1/(\pi \times 10000.00)] \times 0.0$
= 0.0 sec. $\leq$ 0.05 sec.

$$A_{og} = [246 L^3 / (E_s I_g)] (0.1 - t_o) = 0.0 \text{ in.}$$

RIGHT GIRDER CALCULATIONS:

Tributary width = 28.00 ft.
Span: L_g = 35.00 ft.
Equivalent uniform load: w_g = 28.00 x (115.4/2.50) + 27.1
= 1320.8 plf

Transformed moment of inertia:
min(0.2 L_g, 28.00x12/2) + min(0.2 L_g, 28.00x12/2)
= 168.000" (19.911" transformed)



Effective concrete width = 168.000 in. and 84.000 in.
Transformed concrete width = 19.911 in. and 9.955 in.

-- Continued --

Appendix A.4.3

Murray Criterion Calculations

Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 5

Transformed concrete area = 37.333 in.² and 6.222 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 17.392 in. (Above girder c. g.)
Transformed moment of inertia = 4161.1 in.⁴

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{wg L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 4161.1}{(1320.8/12) \times (35.00 \times 12)^4} \right]^{0.5} = 5.79 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi fg = 1.819$$

$$\text{since } t_o = [1/(\pi fg)] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 5.79)] \times 1.07$$

$$= 0.0587 \text{ sec.} > 0.05 \text{ sec.}$$

$$A_{og} = \frac{246 L^3}{\text{Es I}_g} \frac{1}{2 \pi fg} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0063 \text{ in.}$$

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\text{Frequency: } \frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{6.46^2} + \frac{1}{5.79^2}$$

$$f_s = 4.31 \text{ Hz.}$$

$$\text{Amplitude: } A_{os} = A_{ob} + A_{og}/2 = 0.0145 + 0.0063/2 = 0.0176 \text{ in.}$$

Warning:

The floor system does not satisfy the Murray criterion.
Possible solutions are:

- Increase the concrete thickness, which will decrease the natural frequency but will not significantly change the initial amplitude, A_o .
- Change the beam spacing.

Generally, increasing the member stiffness (moment of inertia) will not improve the evaluation.

-- End --

APPENDIX A.5 MODIFIED REIHER-MEISTER CALCULATIONS

Appendix A.5 Modified Reiher-Meister Calculations

A.5.1 HOT-ROLLED SHAPES

Newport V
File # 596
Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

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537 Wisteria Drive
Radford, VA 24141

Date: 01/27/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 1

Span= 30.833 ft		Defaults	
Left Girder:		Table: FLOORVIB.VDT	
Wall			
Left		Right Girder:	
Bm. Span:		W27x84	
0.0 ft		Right	
		Bm. Span:	
		37.667 ft	
Bm Span= 45.83 ft			
Concrete: dc: 5-1/4 in.(total depth)		Office/Residential Activity	
fc': 3.5 Ksi		Design Criterion:Modified Reiher/Meister	
wt.: 115 pcf		System Evaluation:	
Deck Height: 2"		SATISFIES CRITERION	
Loading: Dead: 4 psf		Slightly perceptible vibrations	
Collateral: 12.5 psf			
(V) Live: 8 psf			

VIBRATION ANALYSIS:

Evaluation Criterion: Modified Reiher & Meister
Environment: Office/Residential
Reference: Murray, T.M., "Design to Prevent Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1975,
pages 82-87.

	Section	w, plf	Itr, in4	f, Hz.	Ao in.	Ao f, in/s
Beam	W21x44	724.4	2669.5	3.67	0.0068	0.0249
Left Girder	Wall	1615.1	0.0	10000.	0.0	0.0
Right Girder	W27x84	3027.0	7389.0	6.57	0.0027	0.0175
Bay (Using smaller girder frequency)				3.20	0.0081	0.0259

Evaluation:

The Bay controls.
(Elements having frequencies above 10 Hz. were ignored in the evaluation.)
Vibrations are SLIGHTLY perceptible.
The system SATISFIES THE CRITERION.

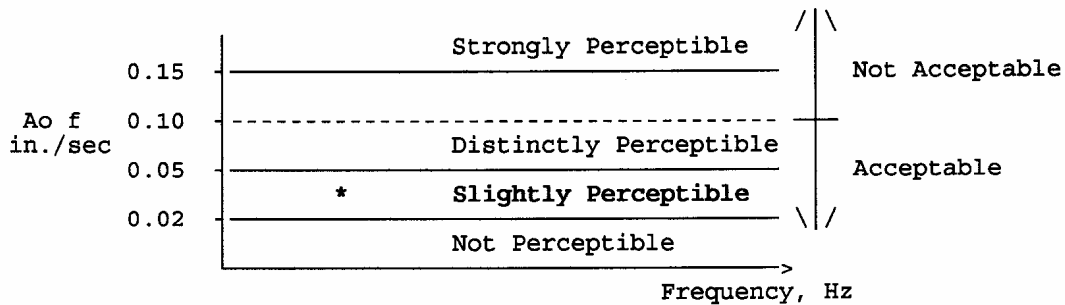
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Appendix A.5.1 Modified Reiher-Meister Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 2



LOADING DATA:

Slab + 1.0 psf Deck = 41.7 psf
Dead loads = 4.0 psf
Collateral loads = 12.5 psf
Live loads = 8.0 psf
Actual beam and girder weights

Tributary width for left girder = $45.83/2 = 22.92$ ft.
Tributary width for right girder = $45.83/2 + 37.67/2 = 41.75$ ft.

CONCRETE/SLAB DATA:

Concrete dc= 5.25 in. fc' = 3.5 Ksi
wt= 115 pcf Ec = 2307 Ksi

Modular ratio, n= 12.57

Deck height: 2 in.
Effective concrete thickness in deck: 1 in.

BEAM CALCULATIONS:

Beam section: W21x44 d= 20.660 in. A= 13.00 in.² Ix= 843 in.⁴

Spacing: S = 123.33 in.
Span: Lb = 45.83 ft.
Uniform load: wb = $(41.7 + 4.0 + 12.5 + 8.0) \times 123.33/12 + 44.0$
= 724.4 plf

Transformed moment of inertia:
Effective concrete width = min(0.4 Lb, S) = 123.330 in.
Effective concrete depth = 3.250 in.
Transformed concrete width = 9.811 in.
Transformed concrete area = 31.886 in.²
Distance to neutral axis = 9.913 in. (Above beam c. g.)
Transformed moment of inertia = 2669.5 in.⁴

-- Continued --

Appendix A.5.1 Modified Reiher-Meister Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 3

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{wb L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 2669.5}{(724.4 / 12) \times (45.83 \times 12)^4} \right]^{0.5} = 3.67 \text{ Hz.}$$

Tee-Beam Amplitude:

$$\alpha = 0.1 \pi fb = 1.153$$

$$\text{since } t_o = [1/(\pi fb)] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 3.67)] \times 0.86$$

$$= 0.0743 \text{ sec.} > 0.05 \text{ sec.}$$

$$Aot = \frac{246 L^3}{Es Ib} \frac{1}{2 \pi fb} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0147 \text{ in.}$$

Number of Effective Tee-Beams:

$$Neff = 2.97 - S/(17.3 de) + L^4/(1.35 Es Itr)$$

$$= 2.97 - 123.33/(17.3 \times 4.25)$$

$$+ (45.83 \times 12)^4/(1.35 \times 29 \times 10^6 \times 2669.49)$$

$$= 2.17$$

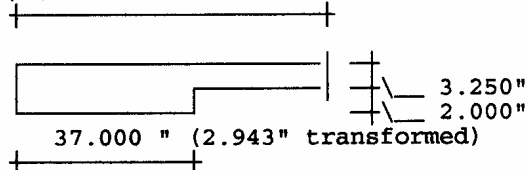
$$\text{Beam Amplitude, } Aob = Aot/Neff = 0.0068 \text{ in.}$$

LEFT GIRDER CALCULATIONS:

Tributary width = 22.92 ft.
Span: Lg = 30.83 ft.
Equivalent uniform load: wg = 22.92 x (724.4/10.28) + 0.0
= 1615.1 plf

Transformed moment of inertia:

$$\min(0.2 Lg, Lb/2) = 73.999 \text{ " (5.887" transformed)}$$



Effective concrete width = 73.999 in. and 37.000 in.
Transformed concrete width = 5.887 in. and 2.943 in.
Transformed concrete area = 19.132 in.2 and 5.887 in.2
Distance to neutral axis = 0.0 in. (Above girder c. g.)
Transformed moment of inertia = 0.0 in.4

-- Continued --

Appendix A.5.1 Modified Reiher-Meister Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 4

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{wg L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 0.0}{(1615.1/12) \times (30.83 \times 12)^4} \right]^{0.5} = 10000.00 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi fg = 0.0$$

$$\text{since } t_o = [1/(\pi fg)] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 10000.00)] \times 0.0$$

$$= 0.0 \text{ sec.} \leq 0.05 \text{ sec.}$$

$$Aog = [246 L^3 / (Es Ig)] (0.1 - t_o) = 0.0 \text{ in.}$$

RIGHT GIRDER CALCULATIONS:

Girder section: W27x84 d= 26.710 in. A= 24.80 in.2 Ix= 2850 in.4

Tributary width = 41.75 ft.

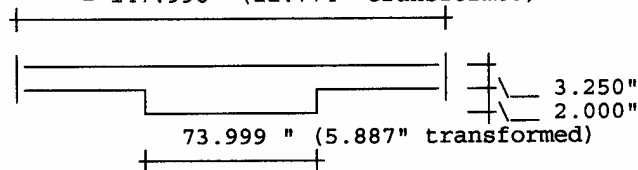
Span: Lg = 30.83 ft.

Equivalent uniform load: wg = 41.75 x (724.4/10.28) + 84.0
= 3027.0 plf

Transformed moment of inertia:

$$\min(0.2 Lg, 45.83 \times 12/2) + \min(0.2 Lg, 37.67 \times 12/2)$$

$$= 147.998" (11.774" \text{ transformed})$$



Effective concrete width = 147.998 in. and 73.999 in.
Transformed concrete width = 11.774 in. and 5.887 in.
Transformed concrete area = 38.264 in.2 and 11.774 in.2
Distance to neutral axis = 10.940 in. (Above girder c. g.)
Transformed moment of inertia = 7389.0 in.4

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{wg L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 7389.0}{(3027.0/12) \times (30.83 \times 12)^4} \right]^{0.5} = 6.57 \text{ Hz.}$$

-- Continued --

Appendix A.5.1 Modified Reiher-Meister Calculations Hot-Rolled Shapes (Cont.)

Newport V
File # 596
Typical Bay

Date: 01/26/03 By: MDB
Job Id: Newport Office Center V
Id: Typ.

Page 5

Girder Amplitude:

$$\begin{aligned}\alpha &= 0.1 \pi f_g = 2.064 \\ \text{since } t_o &= [1/(\pi f_g)] \tan^{-1}(\alpha) \\ &= [1/(\pi \times 6.57)] \times 1.12 \\ &= 0.0542 \text{ sec.} > 0.05 \text{ sec.}\end{aligned}$$

$$A_{og} = \frac{246 L^3}{E_s I_g} \frac{1}{2 \pi f_g} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0027 \text{ in.}$$

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\begin{aligned}\text{Frequency: } \frac{1}{f_s^2} &= \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{3.67^2} + \frac{1}{6.57^2} \\ f_s &= 3.20 \text{ Hz.}\end{aligned}$$

$$\text{Amplitude: } A_{os} = A_{ob} + A_{og}/2 = 0.0068 + 0.0027/2 = 0.0081 \text{ in.}$$

-- End --

Appendix A.5 Modified Reiher-Meister Calculations

A.5.2 HOT-ROLLED GIRDERS WITH JOISTS

Melroe Corporation

File # 563

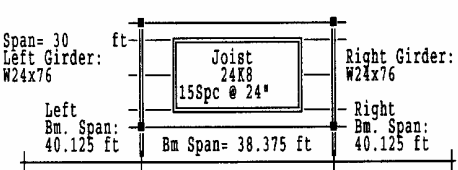
Typical Bay

Knowledge Base FLOORVIB2 Version 1.1B2

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Radford, VA 24141

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 1

		Defaults Table: FLOORVIB.VDT
Span= 30 ft		
Left Girder: W24x76	Joist 24K8 15Spc @ 24"	Right Girder: W24x76
Left Bm. Span: 40.125 ft	Bm Span= 38.375 ft	Right Bm. Span: 40.125 ft
Concrete: dc: 2-3/4 in. (total depth)		Office/Residential Activity
fc': 3 Ksi		Design Criterion: Modified Reiher/Meister
wt.: 145 pcf		System Evaluation:
Deck Height: 5/8"		SATISFIES CRITERION
Loading: Dead: 7 psf		Distinctly perceptible vibrations
Collateral: psf		
(v) Live: 11 psf		

VIBRATION ANALYSIS:

Evaluation Criterion: Modified Reiher & Meister
Environment: Office/Residential
Reference: Murray, T.M., "Design to Prevent Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1975,
pages 82-87.

	Section	w, plf	Itr, in4	f, Hz.	Ao in.	Ao f, in/s
Joist	24K8	108.5	451.4	5.56	0.0099	0.0552
Left Girder	W24x76	2204.4	5664.4	7.12	0.0034	0.0242
Right Girder	W24x76	2204.4	5664.4	7.12	0.0034	0.0242
Bay (Using smaller girder frequency)				4.38	0.0116	0.0509

Evaluation:

The Joist controls.
Vibrations are DISTINCTLY perceptible.
However, the system SATISFIES THE CRITERION.

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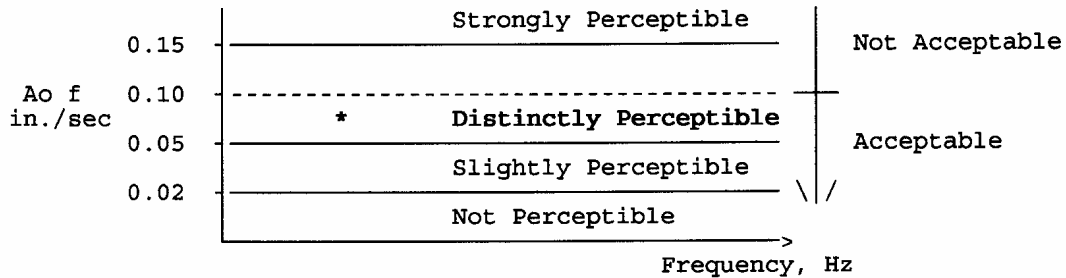
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Appendix A.5.2 Modified Reiher-Meister Calculations Hot-Rolled Girders with Joists (Cont.)

Melroe Corporation
File # 563
Typical Bay

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 2



LOADING DATA:

Slab + 1.0 psf Deck = 30.5 psf
Dead loads = 7.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for girder = $40.13/2 + 38.38/2 = 39.25$ ft.

CONCRETE/SLAB DATA:

Concrete dc= 2.75 in. fc'= 3 Ksi
 wt= 145 pcf Ec= 3024 Ksi

Modular ratio, n= 9.59

Deck height: 0.625 in.
Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 24K8 (11.5 plf)
d= 24.000 in. A= 2.157 in.² Ix= 282 in.⁴ yc= 10.72 in.

Spacing: S = 24 in.
Span: Lb = 38.38 ft.
Uniform load: wb = (30.5 + 7.0 + 0.0 + 11.0) x 24.00/12 + 11.5
 = 108.5 plf

Transformed moment of inertia:
Effective concrete width = min(0.4 Lb, S) = 24.000 in.
Effective concrete depth = 2.125 in.
Transformed concrete width = 2.503 in.
Transformed concrete area = 5.318 in.²
Distance to neutral axis = 9.228 in. (Above beam c. g.)
Transformed moment of inertia = 451.4 in.⁴
 $6 \leq Lj/D=19.19 \leq 24$

-- Continued --

Appendix A.5.2 Modified Reiher-Meister Calculations Hot-Rolled Girders with Joists (Cont.)

**Melroe Corporation
File # 563
Typical Bay**

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 3

$$\begin{aligned} Cr &= 0.9 [1 - e^{(-0.28 L_j/D)}]^{2.8} = 0.89 \\ \tau &= (1/Cr) - 1 = 0.126 \\ \text{Effective moment of inertia} &= 1/[\tau/I_{chords} + 1/I_{comp}] = 375.7 \text{ in.}^4 \end{aligned}$$

$$\begin{aligned} \text{Frequency} &= 1.57 \times \left[\frac{386 E_s I_{tr}}{w_b L^4} \right]^{0.5} \\ &= 1.57 \times \left[\frac{386 E_s \times 451.4}{(108.5/12) \times (38.38 \times 12)^4} \right]^{0.5} = 5.56 \text{ Hz.} \end{aligned}$$

Tee-Beam Amplitude:

$$\begin{aligned} \alpha &= 0.1 \pi f_b = 1.747 \\ \text{since } t_o &= [1/(\pi f_b)] \tan^{-1}(\alpha) \\ &= [1/(\pi \times 5.56)] \times 1.05 \\ &= 0.0602 \text{ sec.} > 0.05 \text{ sec.} \end{aligned}$$

$$A_{ot} = \frac{246 L^3}{E_s I_b} \frac{1}{2 \pi f_b} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0736 \text{ in.}$$

Number of Effective Tee-Beams:

$$\begin{aligned} D_x &= E_c d_e^3 / 12 = 3024 \times 2.44^3 / 12 = 3661 \text{ K-in.} \\ D_y &= E_s I_{tr} / S = 29000 \times 451.4 / 24.00 = 545442 \text{ K-in.} \\ \epsilon &= (D_x/D_y)^{0.25} = 0.29 \\ X_o &= 1.06 \epsilon L_b = 1.06 \times 0.29 \times 460.5 = 139.71 \text{ in.} \\ X &= 24", 48", 72", 96", 120" \end{aligned}$$

$$\begin{aligned} N_{eff} &= 1.0 + 2 \sum \cos(\pi x / (2 x_o)) \\ &= 7.41 \end{aligned}$$

$$\text{Beam Amplitude, } A_{ob} = A_{ot}/N_{eff} = 0.0099 \text{ in.}$$

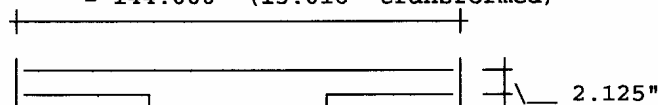
LEFT GIRDER CALCULATIONS:

$$\text{Girder section: } W24 \times 76 \quad d = 23.920 \text{ in.} \quad A = 22.40 \text{ in.}^2 \quad I_x = 2100 \text{ in.}^4$$

$$\begin{aligned} \text{Tributary width} &= 39.25 \text{ ft.} \\ \text{Span: } L_g &= 30.00 \text{ ft.} \\ \text{Equivalent uniform load: } w_g &= 39.25 \times (108.5/2.00) + 76.0 \\ &= 2204.4 \text{ plf} \end{aligned}$$

Transformed moment of inertia:

$$\begin{aligned} &\min(0.2 L_g, 40.13 \times 12/2) + \min(0.2 L_g, 38.38 \times 12/2) \\ &= 144.000" \text{ (15.016" transformed)} \end{aligned}$$



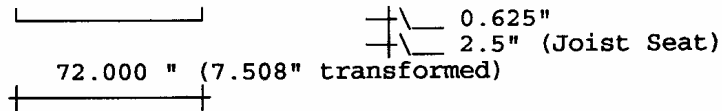
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Appendix A.5.2 Modified Reiher-Meister Calculations Hot-Rolled Girders with Joists (Cont.)

**Melroe Corporation
File # 563
Typical Bay**

Date: 01/27/03 By: MDB
Job Id: Melroe Corp. Building --- West Fargo, N.D.
Id: Typ.

Page 4



Effective concrete width	= 144.000 in.	and	72.000 in.
Transformed concrete width	= 15.016 in.	and	7.508 in.
Transformed concrete area	= 31.908 in. ²	and	4.692 in. ²
Joist seat depth	= 2.5 in.		
Distance to neutral axis	= 9.908 in.	(Above girder c. g.)	
Transformed moment of inertia	= 5664.4 in. ⁴		

$$\begin{aligned}
 \text{Frequency} &= 1.57 \times \left[\frac{386 \text{ Es Itr}}{\text{wg L}^4} \right]^{0.5} \\
 &= 1.57 \times \left[\frac{386 \text{ Es} \times 5664.4}{(2204.4/12) \times (30.00 \times 12)^4} \right]^{0.5} = 7.12 \text{ Hz.}
 \end{aligned}$$

Girder Amplitude:

$$\begin{aligned}
 \alpha &= 0.1 \pi f_g = 2.237 \\
 \text{since } t_o &= [1/(\pi f_g)] \tan^{-1}(\alpha) \\
 &= [1/(\pi \times 7.12)] \times 1.15 \\
 &= 0.0514 \text{ sec.} > 0.05 \text{ sec.}
 \end{aligned}$$

$$A_{og} = \frac{246 \text{ L}^3}{\text{Es I}_g} \frac{1}{2 \pi f_g} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0034 \text{ in.}$$

RIGHT GIRDER CALCULATIONS:

Girder section: W24x76 d= 23.920 in. A= 22.40 in.² I_x= 2100 in.⁴

Same as Left girder.

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\text{Frequency: } \frac{1}{f_s^2} = \frac{1}{f_b^2} + \frac{1}{f_g^2} = \frac{1}{5.56^2} + \frac{1}{7.12^2}$$

$$f_s = 4.38 \text{ Hz.}$$

$$\text{Amplitude: } A_{os} = A_{ob} + A_{og}/2 = 0.0099 + 0.0034/2 = 0.0116 \text{ in.}$$

-- End --

Appendix A.5 Modified Reiher-Meister Calculations

A.5.3 JOIST GIRDERS WITH JOISTS

Garmin International

File # 506

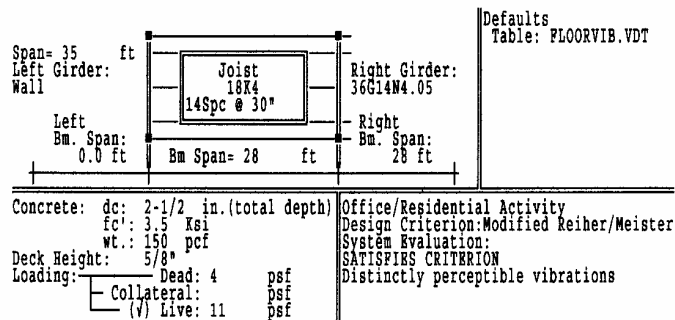
Typical Exterior Bay

Knowledge Base FLOORVIB2 Version 1.1B2

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Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 1



VIBRATION ANALYSIS:

Evaluation Criterion: Modified Reiher & Meister
Environment: Office/Residential
Reference: Murray, T.M., "Design to Prevent Floor Vibrations",
AISC Engineering Journal, 3rd Quarter, 1975,
pages 82-87.

	Section	w, plf	Itr, in4	f, Hz.	Ao in.	Ao f, in/s
Joist	18K4	115.4	183.9	6.46	0.0145	0.0935
Left Girder	Wall	708.5	0.0	10000.	0.0	0.0
Right Girder	36G14N4.	1320.8	4161.1	5.79	0.0063	0.0362
Bay (Using smaller girder frequency)				4.31	0.0176	0.0759

Evaluation:

The Joist controls.
(Elements having frequencies above 10 Hz. were ignored in the evaluation.)
Vibrations are DISTINCTLY perceptible.
However, the system SATISFIES THE CRITERION.

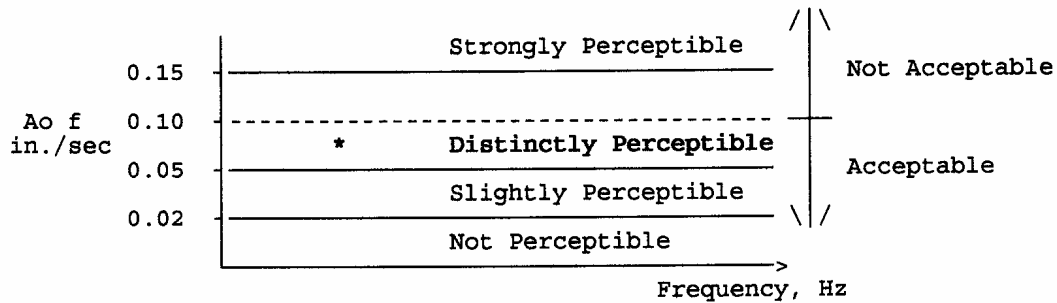
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Appendix A.5.3 Modified Reiher-Meister Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 2



LOADING DATA:

Slab + 1.0 psf Deck = 28.3 psf
Dead loads = 4.0 psf
Collateral loads = 0.0 psf
Live loads = 11.0 psf
Actual beam and girder weights

Tributary width for left girder = $28.00/2 = 14.00$ ft.
Tributary width for right girder = $2 \times (28.00/2) = 28.00$ ft.

CONCRETE/SLAB DATA:

Concrete dc= 2.50 in. $f_c' = 3.5$ Ksi
 wt= 150 pcf $E_c = 3437$ Ksi

Modular ratio, $n = 8.44$

Deck height: 0.625 in.
Effective concrete thickness in deck: 0.3125 in.

JOIST CALCULATIONS:

Joist: 18K4 (7.2 plf)
 $d = 18.000$ in. $A = 1.431$ in.² $I_x = 102$ in.⁴ $y_c = 8.06$ in.

Spacing: $S = 30$ in.
Span: $L_b = 28.00$ ft.
Uniform load: $w_b = (28.3 + 4.0 + 0.0 + 11.0) \times 30.00/12 + 7.2$
 $= 115.4$ plf

Transformed moment of inertia:
Effective concrete width $= \min(0.4 L_b, S) = 30.000$ in.
Effective concrete depth $= 1.875$ in.
Transformed concrete width $= 3.556$ in.
Transformed concrete area $= 6.667$ in.²
Distance to neutral axis $= 8.134$ in. (Above beam c. g.)

-- Continued --

Appendix A.5.3 Modified Reiher-Meister Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 3

Transformed moment of inertia = 183.9 in.⁴
 $6 \leq L_j/D=18.67 \leq 24$
 $Cr=0.9 [1-e^{(-0.28 L_j/D)}]^{2.8} = 0.89$
 $\tau = (1/Cr) - 1 = 0.128$
 Effective moment of inertia = $1/[\tau/I_{chords}+1/I_{comp}] = 149.4 \text{ in.}^4$

$$\text{Frequency} = 1.57 \times \left[\frac{386 E_s I_{tr}}{w_b L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 E_s \times 183.9}{(115.4/12) \times (28.00 \times 12)^4} \right]^{0.5} = 6.46 \text{ Hz.}$$

Tee-Beam Amplitude:

$\alpha = 0.1 \pi f_b = 2.029$
 since $t_o = [1/(\pi f_b)] \tan^{-1}(\alpha)$
 $= [1/(\pi \times 6.46)] \times 1.11$
 $= 0.0548 \text{ sec.} > 0.05 \text{ sec.}$

$$A_{ot} = \frac{246 L^3}{E_s I_b} \frac{1}{2 \pi f_b} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0791 \text{ in.}$$

Number of Effective Tee-Beams:

$D_x = E_c d_e^3 / 12 = 3437 \times 2.19^3 / 12 = 3008 \text{ K-in.}$
 $D_y = E_s I_{tr} / S = 29000 \times 183.9 / 30.00 = 177770 \text{ K-in.}$
 $\epsilon = (D_x/D_y)^{0.25} = 0.36$
 $X_o = 1.06 \epsilon L_b = 1.06 \times 0.36 \times 336 = 128.46 \text{ in.}$
 $X = 30", 60", 90", 120"$

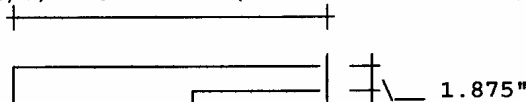
$N_{eff} = 1.0 + 2 \sum \cos(\pi x / (2 x_o))$
 $= 5.47$

Beam Amplitude, $A_{ob} = A_{ot}/N_{eff} = 0.0145 \text{ in.}$

LEFT GIRDER CALCULATIONS:

Tributary width = 14.00 ft.
 Span: $L_g = 35.00 \text{ ft.}$
 Equivalent uniform load: $w_g = 14.00 \times (115.4/2.50) + 62.0$
 $= 708.5 \text{ plf}$

Transformed moment of inertia:
 $\min(0.2 L_g, L_b/2) = 84.000" \text{ (9.955" transformed)}$



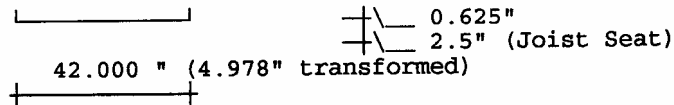
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Appendix A.5.3 Modified Reiher-Meister Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 4



Effective concrete width	=	84.000 in.	and	42.000 in.
Transformed concrete width	=	9.955 in.	and	4.978 in.
Transformed concrete area	=	18.666 in. ²	and	3.111 in. ²
Joist seat depth	=	2.5 in.		
Distance to neutral axis	=	0.0 in.	(Above girder c. g.)	
Transformed moment of inertia	=	0.0 in. ⁴		

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{\text{wg L}^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 0.0}{(708.5 / 12) \times (35.00 \times 12)^4} \right]^{0.5} = 10000.00 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi \text{ fg} = 0.0$$

$$\text{since } t_o = [1 / (\pi \text{ fg})] \tan^{-1}(\alpha)$$

$$= [1 / (\pi \times 10000.00)] \times 0.0$$

$$= 0.0 \text{ sec.} \leq 0.05 \text{ sec.}$$

$$\text{Aog} = [246 \text{ L}^3 / (\text{Es Ig})] (0.1 - t_o) = 0.0 \text{ in.}$$

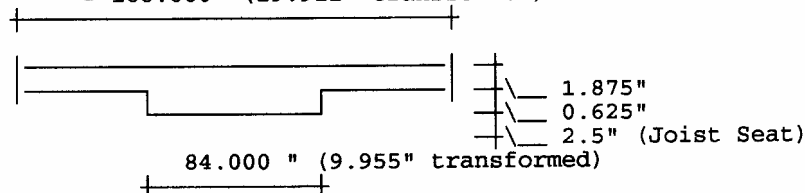
RIGHT GIRDER CALCULATIONS:

Tributary width = 28.00 ft.
Span: Lg = 35.00 ft.
Equivalent uniform load: wg = 28.00 x (115.4/2.50) + 27.1
= 1320.8 plf

Transformed moment of inertia:

$$\min(0.2 \text{ Lg}, 28.00 \times 12 / 2) + \min(0.2 \text{ Lg}, 28.00 \times 12 / 2)$$

$$= 168.000 \text{ " (19.911" transformed)}$$



Effective concrete width	=	168.000 in.	and	84.000 in.
Transformed concrete width	=	19.911 in.	and	9.955 in.

-- Continued --

Appendix A.5.3 Modified Reiher-Meister Calculations Joist Girders with Joists (Cont.)

Garmin International
File # 506
Typical Exterior Bay

Date: 01/27/03 By: MDB
Job Id: Garmin Int. Bldg. -- Olathe, KS
Id: Typ. Ext. #506

Page 5

Transformed concrete area = 37.333 in.² and 6.222 in.²
Joist seat depth = 2.5 in.
Distance to neutral axis = 17.392 in. (Above girder c. g.)
Transformed moment of inertia = 4161.1 in.⁴

$$\text{Frequency} = 1.57 \times \left[\frac{386 \text{ Es Itr}}{wg L^4} \right]^{0.5}$$

$$= 1.57 \times \left[\frac{386 \text{ Es} \times 4161.1}{(1320.8/12) \times (35.00 \times 12)^4} \right]^{0.5} = 5.79 \text{ Hz.}$$

Girder Amplitude:

$$\alpha = 0.1 \pi fg = 1.819$$

$$\text{since } t_o = [1/(\pi fg)] \tan^{-1}(\alpha)$$

$$= [1/(\pi \times 5.79)] \times 1.07$$

$$= 0.0587 \text{ sec.} > 0.05 \text{ sec.}$$

$$Aog = \frac{246 L^3}{Es Ig} \frac{1}{2 \pi fg} \sqrt{2 [1 - \alpha \sin(\alpha) - \cos(\alpha)] + \alpha^2} = 0.0063 \text{ in.}$$

BAY CALCULATIONS:

Using girder with smaller frequency:

$$\text{Frequency: } \frac{1}{fs^2} = \frac{1}{fb^2} + \frac{1}{fg^2} = \frac{1}{6.46^2} + \frac{1}{5.79^2}$$

$$fs = 4.31 \text{ Hz.}$$

$$\text{Amplitude: } Aos = Aob + Aog/2 = 0.0145 + 0.0063/2 = 0.0176 \text{ in.}$$

-- End --

VITA

Michael D. Boice was born in West Palm Beach, Florida on Sunday, March 5, 1978 to Sharan and Gary Boice. He was raised in West Palm Beach and graduated from John. I. Leonard High School in June of 1996. In December 2000, he received his Bachelor of Science degree in Civil Engineering from the University of Florida. In August of 2001, Michael began his graduate career at the Virginia Polytechnic Institute and State University to obtain his Master of Science degree in Civil Engineering. He received his degree with an emphasis in structural design and analysis in February of 2003. From Virginia, Michael went on to bigger and better possibilities in Chicago.

Michael DeLancey Boice

P.S.

There are those that look at things the way they are,
and ask, "Why?" I dream of things that never were,
and ask "Why not?"

--- Robert F. Kennedy

Even a stopped clock gives the right time twice a
day.

--- Unknown