A Study of Computer Modeling Techniques to Predict the Response of Floor Systems Due to Walking

by

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(ABSTRACT)

The possibility of using a commercially available structural analysis program to predict the response of a floor system due to walking excitation as given in AISC Design Guide 11, *Floor Vibrations Due to Human Activity* (Murray, et al., 1997) was explored. This research included ideal floors that did not have measured values as well as several case study floors that do have measured values for the fundamental frequency.

First, multiple model set-ups and loading protocols are applied to the ideal floors and the results compared to results from the Design Guide procedure. A recommendation of the best combination of a model set-up and loading protocol that best matches the Design Guide procedure results is made. Then, case study floors are modeled with the recommended model set-up and loading protocol, and the results compared to the results from the Design Guide procedure and to measured fundamental frequencies. The peak accelerations are also compared to subjective evaluations as to the acceptability of the system.

Next, multiple systems were analyzed using five different modeling techniques, including the Design Guide Method, an alteration of the Design Guide Method, the Rayleigh Method, the Analytical Method, and the structural analysis program method, in an attempt to determine the source of discrepancies between the structural analysis program method and the Design Guide method.

Finally, conclusions are drawn regarding the structural analysis program procedure as well as possible sources of differences. In general, the structural analysis program procedure reliably predicts the fundamental frequency of a floor system, but does not predict the Design Guide peak acceleration under dynamic loading. The difference in the effective mass of a system between the two methods is a source of discrepancy.

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CHAPTER I
INTRODUCTION AND LITERATURE REVIEW

1.1 INTRODUCTION

The purpose of this study is to investigate the correlation between the procedure used to predict the response of a floor system due to walking utilizing a commercially available structural analysis program with the procedure developed by Allen and Murray (1993) which is outlined in American Institute of Steel Construction (AISC) Design Guide 11 *Floor Vibrations Due to Human Activity* (Murray, et al, 1997), hereafter known as Design Guide.

1.2 AISC DESIGN GUIDE 11

1.2.1 Overview

The method outlined in Design Guide is divided into two sets of calculations. One set estimates the frequencies of the beams, girders, and floor system. With the exception of cantilever sections, members are assumed to be simply supported. The other set of calculations estimates the peak acceleration of the floor system due to walking. An effective mass must be calculated and then used with a dynamic load of a specific magnitude oscillating at the fundamental frequency of the floor system.

1.2.2 Frequency

The fundamental frequency of a joist or beam is given in the Design Guide as:

$$f_j = 0.18\sqrt{\frac{g}{\Delta_j}} \tag{1.1}$$

where g is the acceleration due to gravity and Δ_j is the deflection of the member under an estimated loading. Since the members are assumed to be simply supported with uniform loading, the maximum deflection is:

$$\Delta_j = \frac{5w_j L_j^4}{384 E_s I_j} \quad \textbf{(1.2)}$$

where w_j is the uniform load on the element, L_j is the length of the element, E_s is the modulus of elasticity for steel, and I_j is the composite moment of inertia. For hot-rolled shapes, the moment of inertia used is the full composite moment of inertia. However, an effective moment of inertia must be used for a joist due to significant web shear deformations and joist eccentricity. For joists, the effective composite moment of inertia is:

$$I_{eff} = \frac{1}{\dfrac{\gamma}{I_{chords}} + \dfrac{1}{I_{comp}}} \quad \textbf{(1.3)}$$

Here, I_{chords} is the moment of inertia of the joist's chords, I_{comp} is the fully composite moment of inertia of the section, and γ is:

$$\gamma = \frac{1}{C_r} - 1 \quad \textbf{(1.4)}$$

In Equation 1.4, C_r is a modification factor proposed by Band and Murray (1996) to estimate the effective moment of inertia of a joist or joist-girder. Their study showed that a reduction factor was needed because of the shear deformations and joint eccentricity in the web members of joists and joist-girders. Equation 1.5 applies to joists with angled web members and Equation 1.6 applies to joists with rod web members. For both equations, L is the length of the member, in., and D is the nominal depth, in.

$$C_r = 0.90(1 - e^{-0.28(L/D)})^{2.8} \quad \text{for } 6 \le L/D \le 24 \tag{1.5}$$

$$C_r = 0.721 + 0.00725(L/D) \quad \text{for } 10 \le L/D \le 24 \tag{1.6}$$

The frequency calculation for a girder is the same as for a beam or joist if the girder does not support joists. In the case that the girder supports joists, the moment of inertia must be reduced because the joist seats are not sufficiently stiff to allow the use of the full composite moment of inertia. The effective moment of inertia of the girder is given as:

$$I_g = I_{nc} + (I_c - I_{nc})/4 \tag{1.7}$$

where I_{nc} is the moment of inertia of the base steel and I_c is the composite moment of inertia of the girder. For joist-girders, the non-composite moment of inertia is given by:

$$I_{nc} = C_r I_{chords} \tag{1.8}$$

The fundamental frequency of the floor system is predicted by using Equation 1.9. In the equation, Δ_j is the mid-span deflection of the beam or joist, Δ_g is the modified mid-span deflection of the girder, and g is the acceleration due to gravity.

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} \tag{1.9}$$

1.2.3 Walking Accelerations

The second set of calculations in the Design Guide method is calculation of the predicted peak acceleration due to a person walking across the floor. A harmonic forcing function is used to model the worst case scenario of a person repeatedly stepping at the midpoint of the floor with a frequency that is a harmonic of the fundamental frequency of the floor system. The forcing function is given by:

$$F_i = P\alpha_i \cos(2\pi i f_{step} t) \quad \textbf{(1.10)}$$

where P is a person's weight, taken as 157 lb, i is the harmonic multiple of the step frequency, f_{step} is the step frequency, and α_i is the dynamic coefficient from Table 1.1.

Table 1.1 – Common Forcing Frequencies, *f*, and Dynamic Coefficients, α_i

(From the Design Guide)

Harmonic, *i*	Person Walking	
	f, Hz	α_i
1	1.6 – 2.2	0.50
2	3.2 – 4.4	0.20
3	4.8 – 6.6	0.10
4	6.4 – 8.8	0.05
α is the peak sinusoidal force divided by weight of person(s)		

A steady state response function that predicts the peak acceleration is given by:

$$\frac{a}{g} = \frac{R\alpha_i P}{\beta W} \cos(2\pi i f_{step} t) \quad \textbf{(1.11)}$$

where a/g is the ratio of the floor acceleration to the acceleration due to gravity, R is a reduction factor, β is the modal damping ratio, and W is the effective weight of the floor. R is recommended as 0.7 for footbridges and 0.5 for structures with two-way action.

For design, Equation 1.11 reduces to (Allen and Murray, 1993):

$$\frac{a_p}{g} = \frac{P_0 e^{-0.35 f_n}}{\beta W} \tag{1.12}$$

where a_p/g is the estimated peak acceleration, P_0 is a constant force equal to 65 lb for floors and 92 lb for footbridges, and f_n is the fundamental frequency of the floor system.

The effective weight of a floor system is a combination of the effective weights of the beam panel and the girder panel. In general, the effective weight of a panel is given as:

$$W = wBL \tag{1.13}$$

where W is the effective panel weight, w is the weight per unit area of the panel, B is the effective width of the panel, and L is the length of the panel.

The equation to calculate the effective width of a joist panel is:

$$B_j = C_j(D_s/D_j)^{1/4} L_j \leq {}^2/_3 \text{ Building Floor Width} \tag{1.14}$$

where B_j is the effective width of the joist panel, C_j is 2.0 for most joists and 1.0 for joists or beams parallel to an interior edge, D_s is transformed moment of inertia per unit width, D_j is the effective moment of inertia of the joist per unit of width, and L_j is the length of the joist.

For girders, the effective width is given by:

$$B_g = C_g(D_j/D_g)^{1/4} L_g \leq {}^2/_3 \text{ Building Floor Length} \tag{1.15}$$

where B_g is the effective width of the girder panel, C_g is 1.6 for girders supporting joists connected to the girder flange and 1.8 for girders supporting beams connected to the girder web, D_j is the same as previous, D_g is the effective moment of inertia of the girder per unit width, and L_g is the length of the girder.

If $B_j > L_g$, the mid-span girder deflection is reduced by $L_g / B_j \geq 0.5$. Otherwise Δ_g remains the same.

The effective weight for the combined mode of vibration is a function of the relative stiffness of the beam or joist to the girder and the effective weight of the beam or joist panel and the girder panel. The effective weight for the combined mode is:

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g \qquad \textbf{(1.16)}$$

If a beam, joist, or girder is continuous over a support and an adjacent span is at least seven-tenths of the center span, the effective weight of that panel is increased 50% to account for continuity. It is acceptable to assume that shear connections for hot-rolled members provide enough stiffness to make the continuity adjustment. However, joists that are only connected by the top chord and girders that frame into columns are excluded from the continuity increase.

After the predicted peak acceleration is calculated, it must be compared to a standard value,

$$\frac{a_p}{g} = \frac{P_0 e^{-0.35 f_n}}{\beta W} \leq \frac{a_0}{g} \qquad \textbf{(1.17)}$$

Here, a_0/g is a limit that can be found in Figure 1.1 which is based on a scale proposed by the International Standards Organization (ISO 1989) and was incorporated into the Design Guide. The Design Guide recommended acceleration limits correspond to what people consider annoying vibrations for the activity which they are engaged. For example, people in offices find vibrations with a peak acceleration of 0.5%g to be annoying. However, a person engaged in an activity such as dancing can accept vibrations as much as 10 times the magnitude they can in an office environment.

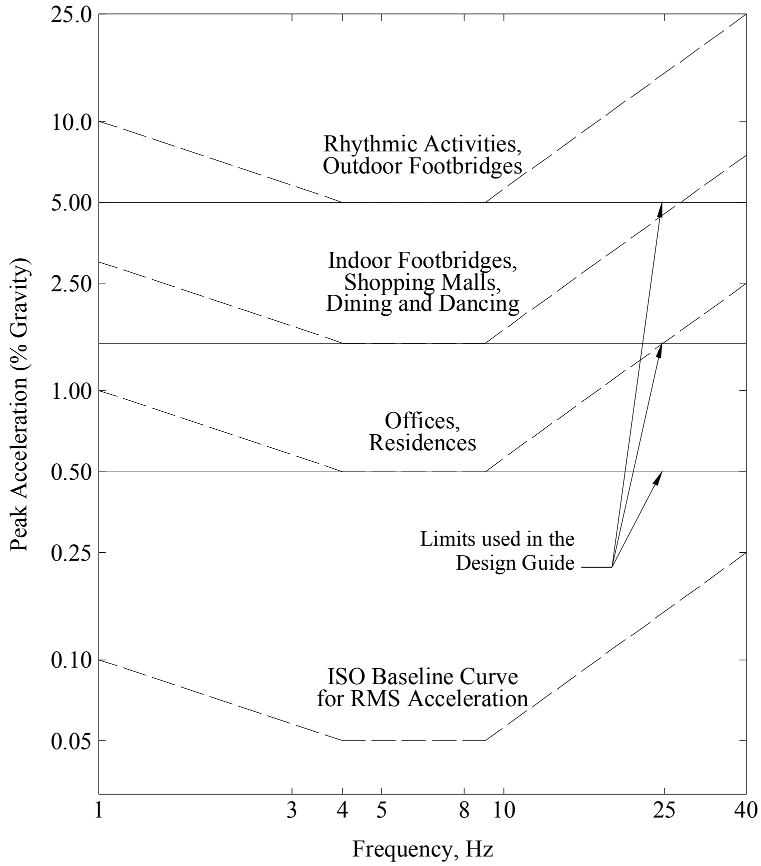


Figure 1.1 - Recommended Peak Accelerations

1.3 OTHER CURRENT RESEARCH

Many individuals have completed research on floor vibrations, some of which is directly related to the research presented in this paper. Kitterman (1994) investigated the behavior of steel joists and joist-girders. He developed equations to calculate the effective moment of inertia of framing members based on test results and finite element modeling. Rottmann (1996) investigated the use of tuned mass dampers for retrofitting existing floor systems that exhibit vibration problems. Band (1996) researched joist and joist-girder supported floors and helped develop the reduction factor used in calculating the effective moment of inertia for a joist. Beavers (1998) investigated the use of a structural analysis program to model single-bay joist-supported floors with the intent of predicting the fundamental frequency of the floors. Sladki (1999) investigated the use of a structural analysis program to predict the natural frequency of a floor system as well as its predicted peak acceleration. It was found that using a structural analysis program to model a floor system is an effective tool for predicting the natural frequency of a floor system, but insufficient for predicting the peak acceleration. He developed many of the modeling conventions used in this study. Warmoth (2002) investigated the effect of different joist seats on the girder moment of inertia and proposed a new calculation for the girder moment of inertia based on the joist seat type. Jackson (2002) examined the properties of castellated beams with respect to vibrations. Boice (2003) explored different methods of predicting the natural frequency and response of a floor system in comparison to actual measured data. He concluded that the procedure outline in the Design Guide is an effective method to predict the fundamental frequency and peak acceleration of a floor system. Ritchey (2003) explored the effectiveness of tuned-mass-dampers that incorporate semi-active magneto-rheological dampers as an effective means to reduce floor vibrations.

1.4 NEED FOR RESEARCH

A variety of methods exist to predict the response of a floor system indicating the complexity of floor vibration analysis. Also, each method requires numerous assumptions about the floor system, making exceptions to each of the available methods. This study investigated the use of a structural analysis program to predict the fundamental frequency of a floor system as well as its dynamic response due to walking without regards to the regularity of beam spacing, material properties, damping properties, and other limitations that exist in the current acceptable methods. Current methods used in predicting the dynamic properties of a floor system treat the system as a single degree of freedom system. However, real floor systems are much more complex than a single degree of freedom system can approximate.

The results from the structural analysis program analyses are compared to the results of analyses from the method outlined in the Design Guide as well as measured data where applicable. Chapter II presents the procedure used to model a floor system in a structural analysis program. The equipment and testing methods used in the in-situ measurements are also discussed. In Chapter III, different model types and loading protocols for the structural analysis program are discussed as well as the results of these analyses. Also discussed in Chapter III is the application of the modeling procedure to actual buildings and the comparison of results from the analyses and measured data. Chapter IV provides a description of the differences between the Design Guide method, an ideal analytical method, and the structural analysis program method. In Chapter V, conclusions are made as well as recommendations for further research. Appendix A contains a detailed example of a floor system modeled using the Design Guide procedure. Appendix B contains a detailed example of a floor system modeled with the structural analysis program procedure. Appendix C includes an example set of calculations for the Analytical Method, and Appendix D has an example set of calculations for the Rayleigh Method.

CHAPTER II
STRUCTURAL ANALYSIS PROGRAM MODELING AND IN-SITU MEASUREMENT TECHNIQUES

2.1 INTRODUCTION

This chapter gives an overview of the modeling techniques used to investigate the abilities of a commercially available structural analysis program to predict the fundamental frequency of a floor system as well as the dynamic response of the floor system due to walking. Also, presented are the methods and equipment used to make in-situ measurements.

2.2 STRUCTURAL ANALYSIS PROGRAM MODELING

2.2.1 Computer Program

The commercially available structural analysis program SAP2000 Nonlinear Version 8.2.3, hereafter SAP2000, available from Computers and Structures, Inc. was used to model the floor systems (Computers and Structures, 2002). SAP2000 can perform an array of complex structural analyses, including linear and nonlinear static and dynamic analyses, within a graphical user interface that is easy to learn and use. The results of the analyses are displayed in tabular format making them easy to comprehend and analyze.

2.2.2 Elements Used

To model a floor system in SAP2000, two types of elements are used: frame and shell elements. SAP2000 defines a frame element as a two-node element that has three translational and three rotational degrees of freedom at each node. The frame element is used to model the beams/joists and girder members of a floor system. The shell element is used to model the concrete slab. For the modeling of this study, all elements are located in the same plane.

2.2.3 Modeling Procedure

Materials. For the modeling of floor systems, two materials are used. One material used was the predefined material named STEEL that can be found in SAP2000. This material was assigned to any frame elements present in the model. The other material, VIBCON, is a user defined material and is assigned to the shell elements in the model. Most of the properties of this material correspond to the concrete materials in the deck. However, SAP2000 calculates the modal response of a system independent of any loads present. Sladki (1999) demonstrated that the unit weight, W, and mass, M, of the material VIBCON is given by:

$$W = \left[\left(\frac{d + d_r/2}{12}\right)w_c + w_d + w_{deck} + w_l + w_{coll}\right]\left(\frac{12}{d}\right)\left(\frac{1}{1{,}728{,}000}\right) \qquad \textbf{(2.1)}$$

$$M = \frac{W}{386} \qquad \textbf{(2.2)}$$

In these formulae, W is the unit weight of the material (kips/in^3), d is the depth of concrete above the metal deck (in.), d_r is the height of the metal deck (in.), w_c is the unit weight of concrete (pcf), w_d is the actual superimposed dead load (psf), w_{deck} is the weight of the deck (psf), w_l is the actual live load (psf), and w_{coll} is the collateral loading (psf). Collateral loading is any load that is an addition to typical dead and live loads. The units for the unit mass used in SAP2000 are kip-s^2/in^4. The modulus of elasticity of VIBCON was set to the dynamic modulus of elasticity of the concrete as defined in the Design Guide, e.g. 1.35 times the static modulus of elasticity of the concrete. Also, the Poisson's ratio for the material, ν, was set as 0.2.

Frame Sections. A minimum of two frame sections must be defined: one for the beam or joist and one for the girder. If more than one beam or girder section exists in the model, a new frame section must be defined. For each section, the cross sectional area is defined as the cross sectional area of the hot-rolled section, joist, or joist girder. The torsional constant as well as the weak axis moment of inertia is set to 1.0 for all sections.

The shear areas for all directions are set to zero because the members are relatively long and slender and shear deformations have little to no effect. The strong axis moment of inertia is set as the transformed moment of inertia of the section as defined by the Design Guide less the moment of inertia of the slab about its own centroid. The moment of inertia of the slab must be subtracted from the frame section moment of inertia because SAP2000 will automatically account for the moment of inertia of the shell element during the analysis and subtracting it will eliminate any errors due to double counting the slab moment of inertia. The moment of inertia to be used in the SAP2000 is given by:

$$I_{SAP} = I_{comp} - \tfrac{1}{12} b_e d^3 \tag{2.3}$$

In Equation 2.3, I_{SAP} is the moment of inertia to be used in the SAP2000 model, I_{comp} is the composite moment of inertia of the framing member, b_e is the effective width of the concrete slab, and d is depth of the concrete above the deck.

Area Section. An area section named SLAB was also created. The VIBCON material is assigned to the section with a material angle of zero. The area type is set to shell and the bending and membrane thickness are both equal to the depth of concrete above the metal deck. The type for the area section is set as thin-plate shell.

Frame Elements. In agreement with the Design Guide, all frame elements are assumed to be simply supported; therefore both ends of all frame sections have major and minor moment releases. The frame elements for the girders are defined to go between column-points and are assigned the proper frame section. The beams/joists are drawn using the Quick Draw Secondary Beams command in SAP2000. This command allows all beams within a bay to be drawn at once as opposed to one member at a time. SAP2000 will automatically subdivide the girder frame elements at intersecting points with other frame elements. To ensure that SAP2000 will give correct results for the modal analysis case, the frame members need to be further subdivided. The best method for this is to assign the frame members an Automatic Frame Subdivide with a minimum number of segments. The typical minimum used for this study is 18 or 20 subdivisions. SAP2000 will automatically subdivide the frame element into a minimum number of segments that are specified when an analysis is ran, however when the model is "unlocked" so that changes may be made, the members are treated as a single member (Computers and Structures, 2002).

Area Elements. An area element must be drawn between column-points. The area has the SLAB area section assigned to it. Similar to the frame members, the area elements need to be meshed with the Automatic Area Mesh command. The mesh of the area needs to correspond to the number of minimum segments specified for the frame members. The area mesh must be even number by even number to ensure the center of the bay fits within the mesh.

Center Node. A node must be drawn at the center of the bay. If the subdivision of frames and the mesh of the area elements match, the node does not need to be attached to anything. This node is needed to apply the loading protocol and to obtain the results for acceleration and displacement. If the node happens to fall in between two frame members, the loading protocol is applied at two nodes located on the closest two frame members to the center of the bay. The loading is distributed evenly between the two nodes. This is done to eliminate any local distortion attributed to the shell element.

Restraints. At every point where there is a column, the node is restrained in all translational degrees of freedom.

2.2.4 Loading Protocols and Time History Analyses

Once the model was created, two different static loads were applied. One was the dead load of the structure. This was used to check deflections and the total mass of the system. The other static load was a 1 lb (or 0.5 lb if divided between two nodes) vertical load. This load, along with different time history functions, was used for all time history analyses. A total of seven time history functions were used in this study. Tables 2.1 and 2.2 list the time history functions used. The first was developed by Allen (1990) who showed that a point load oscillating at a natural frequency of a system would give the same response as predicted by Equation 1.12. The time history function used is:

$$\mathrm{P(t)} = 65e^{-0.35 f_n} \cos(2\pi f_n t) \qquad \textbf{(2.4)}$$

where f_n is the fundamental frequency of the floor system, Hz, and t is time, s. The second through fifth time history functions are Fourier series functions that are presented in the Design Guide. The third and fifth functions are scaled versions of the second and fourth functions, respectively. Also, the fourth and fifth functions are the second and third functions with phase shifts. The final two functions used are time history functions that are presented in Vibration Problems in Structures: Practical Guidelines (IBK 1995). The last loading function is a scaled version of the sixth. The reduction factor used in the third, fifth, and seventh loading functions is 0.50. This reduction accounts for full steady-state resonant motion is not achieved for walking and that the walking person and the person annoyed are not simultaneously at the point of maximum modal displacement. The Design Guide recommends, for a two-way system, that the Fourier series time history functions be reduced to 50% of their original magnitudes. For all of the Fourier series functions, f_{step} is a harmonic of the fundamental frequency of the floor system such that $f_n = if_{step}$.

Table 2.1 – Loading Protocols

Loading Function	Loading Protocol Name	Loading Protocol
1	DG Function Loading	$P(t) = 65e^{-0.35f_n}\cos(2\pi f_n t)$
2	DG Fourier Series Loading	$P(t) = \Sigma 157\alpha_i \cos(2\pi i f_{step} t)$
3	DG Fourier Series Loading (50%)	$P(t) = \Sigma 78.5\alpha_i \cos(2\pi i f_{step} t)$
4	DG Fourier Series Loading with Phase Shift	$P(t) = \Sigma 157\alpha_i \cos(2\pi i f_{step} t - \phi_i)$
5	DG Fourier Series Loading with Phase Shift (50%)	$P(t) = \Sigma 78.5\alpha_i \cos(2\pi i f_{step} t - \phi_i)$
6	IBK Fourier Series	$P(t) = \Sigma 180\alpha_i \sin(2\pi i f_{step} t - \phi_i)$
7	IBK Fourier Series (50%)	$P(t) = \Sigma 90\alpha_i \sin(2\pi i f_{step} t - \phi_i)$

Table 2.2 – Loading Protocol Coefficients

Harmonic i	DG Fourier Series Loading			DG Fourier Series Loading with Phase Shift			IBK Fourier Series		
	f_{step}, Hz	α_i	ϕ_i	f_{step}, Hz	α_i	ϕ_i	f_{step}, Hz	α_i	ϕ_i
1	1.6-2.2	0.50	-	1.6-2.2	0.50	0	1.6-2.4	0.50	0
2	3.2-4.4	0.20	-	3.2-4.4	0.20	$\pi/2$	3.2-4.8	0.10	$\pi/2$
3	4.8-6.6	0.10	-	4.8-6.6	0.10	$\pi/2$	4.8-7.2	0.10	$\pi/2$
4	6.4-8.8	0.05	-	6.4-8.8	0.05	$\pi/2$	-	-	-

Depending on the purpose of the model, up to seven dynamic analyses were performed for each floor. Every model in Chapter III had an analysis case in which the loading was the Design Guide Function loading protocol. However, the other six loading protocols were applied only to the floors listed in Section 3.2.

For each floor, the MODAL analysis case was performed to obtain the fundamental frequency of the system. Since the Design Guide only considers the fundamental mode of vibration, just the fundamental mode is considered in the computer modeling techniques. Therefore, the MODAL analysis case must be modified to only find the fundamental mode.

For each dynamic analysis that is to be performed, a new analysis case is created. The parameters for each load case are listed in Table 2.3. The loads applied for the dynamic analysis are a 1 lb static load and the function is the corresponding time history function as listed in Tables 2.1 and 2.2 with a maximum time step of 0.01 s.

Table 2.3 – Dynamic Analysis Case Parameters

Parameter	Value
Analysis Case Type	Linear
Initial Conditions	Zero Initial Conditions
Time History Type	Modal
Time History Motion Type	Transient
Modal Analysis Case	MODAL
Number of Output Time Steps	10,000
Output Time Step Size	0.01
Modal Damping	Value Estimated by Measurement Team

2.3 IN-SITU MEASUREMENTS

Results from the Design Guide method and from the structural analysis program method were compared to measured data when possible. The measured values can be obtained from the publication by Boice and Murray (2003). A variety of different tests was conducted on each floor. During the tests, the response of the floor to different loading was measured. For each floor, a subjective evaluation of the floor was made.

Testing Equipment. A series of tests were performed by a measurement team to measure the fundamental frequency of the floor and the dynamic response of the floor. The data was collected with an Ono Sokki CF-1200 Handheld FFT Analyzer. The analyzer was connected to a seismic accelerometer, model 393C manufactured by PCB Piezotronics. For each test, the data from the accelerometer was recorded on a memory card in the analyzer. The analyzer then performed a Fast Fourier Transformation on the data and created a frequency response spectrum that was also stored on the memory card. All of the data was then transferred from the memory card to a personal computer for further analysis.

Floor Excitations. For each floor, as many as six different tests, each with a different excitation, were executed. Not all excitations were recorded for all floors. In general, the accelerometer was placed as close to the center of the bay as possible since that is the location of maximum response. The tests were generally performed as close to the accelerometer as practical. The ambient vibration of the floor was usually measured first. For this excitation, the measurement team simply measured the ambient vibration in the floor system at the time of testing. The fundamental frequency of the floor system can easily be determined from the results of the ambient test. The other types of excitation included walking parallel to the beams/joists, walking perpendicular to the beams/joists, a heel drop, walking in place, and bouncing or rhythmic, excitation. Each of these tests gives additional data that can be compared to other procedures in evaluating the floors. Since the input forcing functions are not known and can have variations, the results from the latter excitations cannot accurately be compared to the structural analysis program results. Therefore, the only excitation of interest for this study is the ambient excitation to get the fundamental frequency of the floor system.

Subjective Evaluations. In addition to having quantitative tests performed on each floor, a qualitative test was performed. For the qualitative test, members of the measurement team were asked to indicate how noticeable, in their expert opinions, the vibrations from a person walking were as well as the acceptability of the floor. The opinions of any present occupants of the building during testing were included.

CHAPTER III
TEST CASES AND COMPARISON OF RESULTS

3.1 INTRODUCTION

The structural analysis program model procedure, which is outlined in Chapter II, was used to model two sets of floor systems. The first set was five floors in which every bay is identical. This set was used to determine the best model set-up and loading protocol to use in SAP2000. The second set of floors modeled consisted of a number of floors that exist throughout the United States that have measured data for the fundamental frequency as well as a subjective analysis. The next section describes the first set of floors and compares the results of the structural analysis program model procedure analyses with the results of the Design Guide procedure analyses. The five different model set-ups used are also discussed in that section. The rest of the chapter discusses the second set of floor systems and a comparison of the results from the structural analysis program model procedure analyses, the Design Guide procedure analyses, and the measured data.

3.2 IDEAL FLOOR SYSTEMS

3.2.1 Floor System Descriptions

Five floors were analyzed for this part of the study, Floors A through E. Each of the floor systems consists of five bays in each direction that are the same in size, framing members, and loading. Detailed floor plans for Floors A through E are found in Figures 3.1 through 3.5, respectively. The data for the concrete properties, actual loading information, and damping information are found in Table 3.1. The center bay of each floor is the bay being checked in the analyses.

Each of the floor systems were designed for office occupancy with a 50 psf unreduced live load, a 20 psf partition load, and an 8 psf superimposed dead load. The serviceability limit that the live load deflection must be less than or equal to the span divided by 360 was implemented in the design. The floors also have exterior walls that sufficiently stiffens the spandrel beams and girders such that, for floor vibration purposes, the spandrel moment of inertia is infinity ($I_{exterior} = \infty$ in^4). Table 3.2 lists the moment of inertia for each of the framing members used in the Design Guide procedure (I_{comp}) and the computer analysis program modeling procedure (I_{SAP}).

Table 3.1 – Floor System Information

Floor System	Concrete Depth, d (in.)	f_c' (ksi)	w_c (pcf)	Deck Height, d_r (in.)	Dead* Load (psf)	Live Load (psf)	Damping, β
Floor A	6 ½	3.0	110	3	4	11	0.03
Floor B	5 ¼	4.0	110	3	4	11	0.03
Floor C	5 ¼	4.0	110	2	4	11	0.03
Floor D	5 ¼	4.0	110	2	4	11	0.03
Floor E	5 ¼	4.0	145	2	4	11	0.03
*Dead load in addition to weight of slab and supporting members.							

Table 3.2 – Floor System Stiffness Properties

Floor System	Beams			Girders		
	Section	I_{comp} (in^4)	I_{SAP} (in^4)	Section	I_{comp} (in^4)	I_{SAP} (in^4)
Floor A	W18x35	1950	1920	W21x50	3623	3575
Floor B	W12x14	437	430	W18x35	1882	1867
Floor C	W14x22	804	779	W18x35	1770	1740
Floor D	W16x26	1192	1155	W21x50	3279	3235
Floor E	W21x44	2943	2894	W27x84	8552	8474

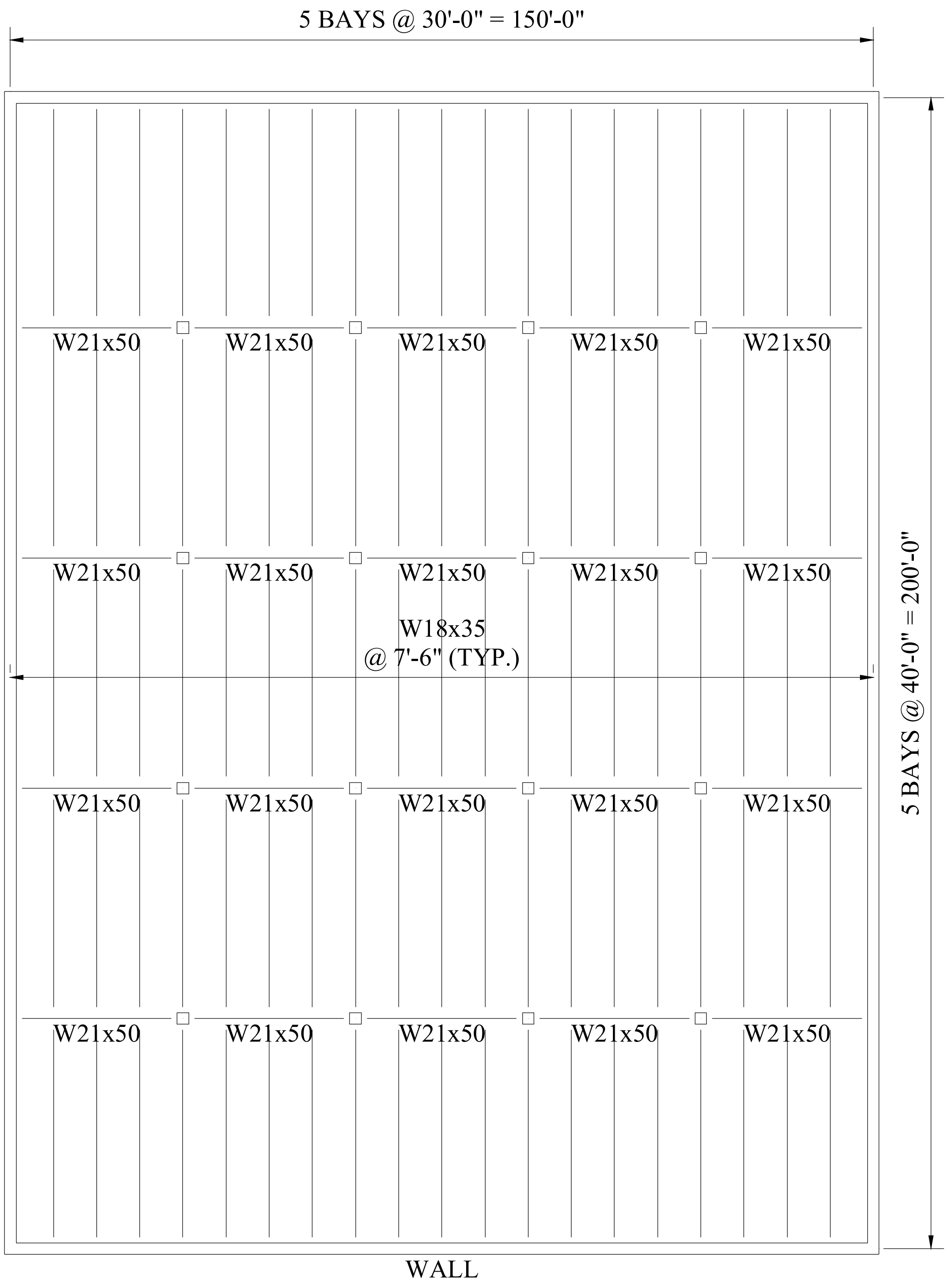


Figure 3.1 – Floor A Detailed Floor Plan

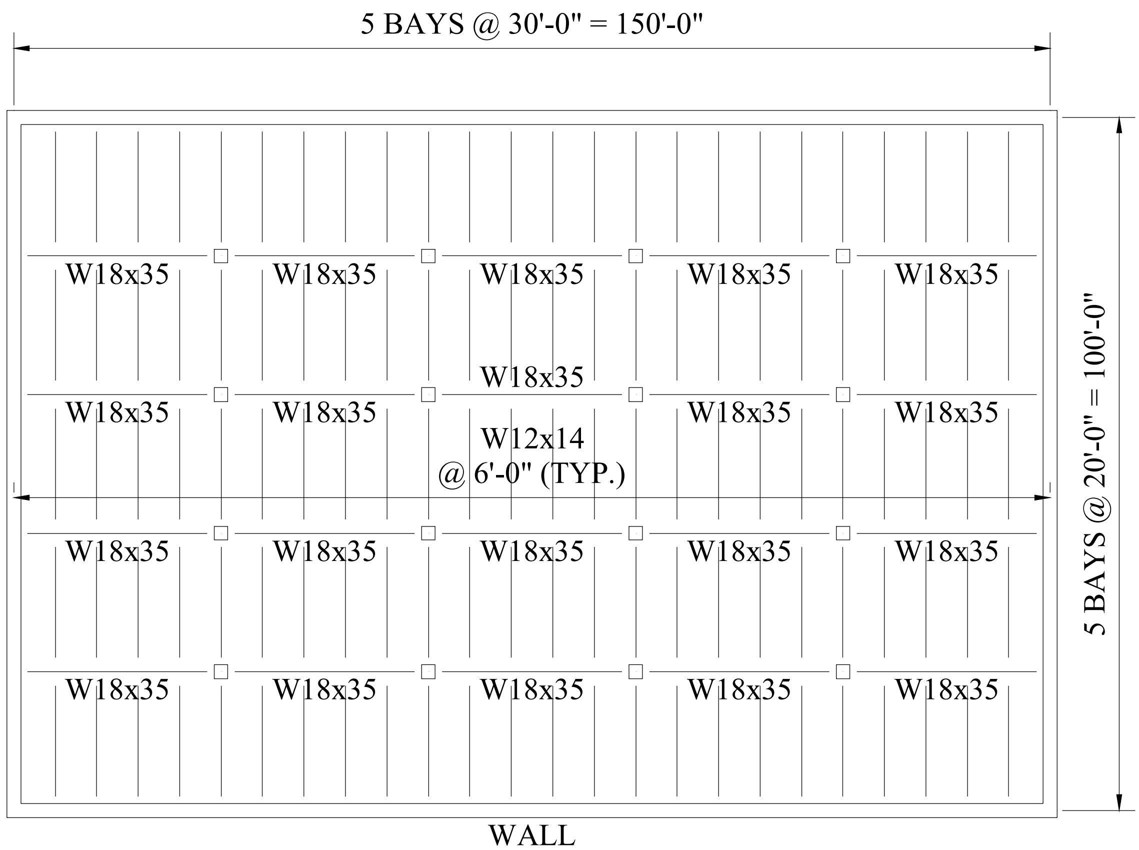


Figure 3.2 – Floor B Detailed Floor Plan

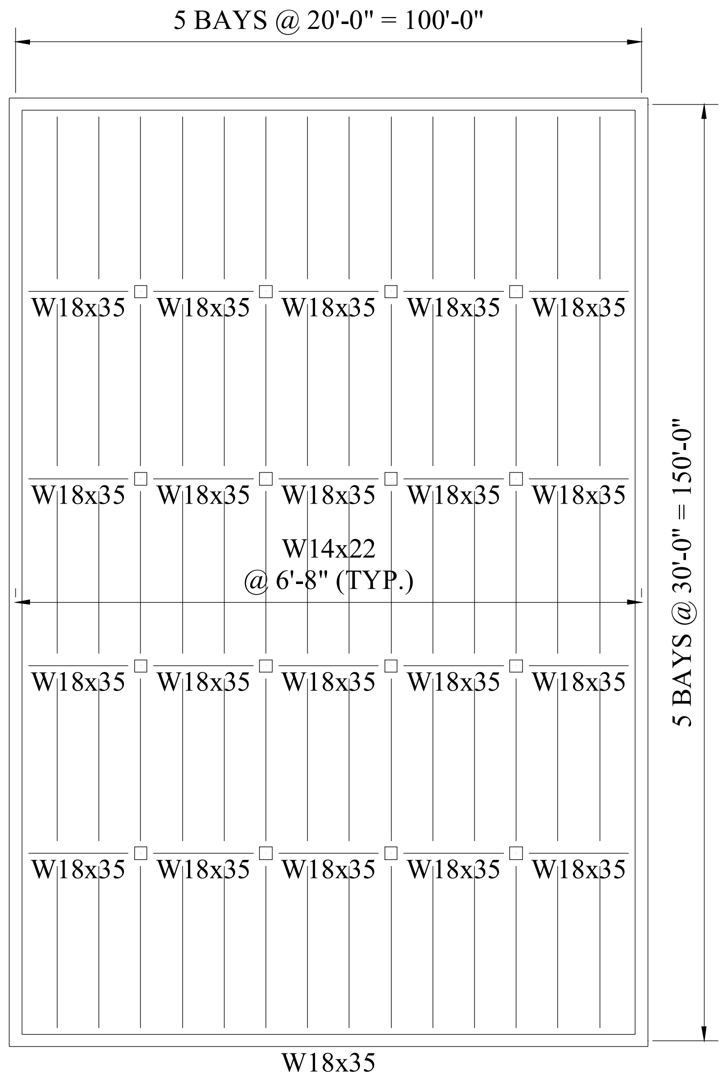


Figure 3.3 – Floor C Detailed Floor Plan

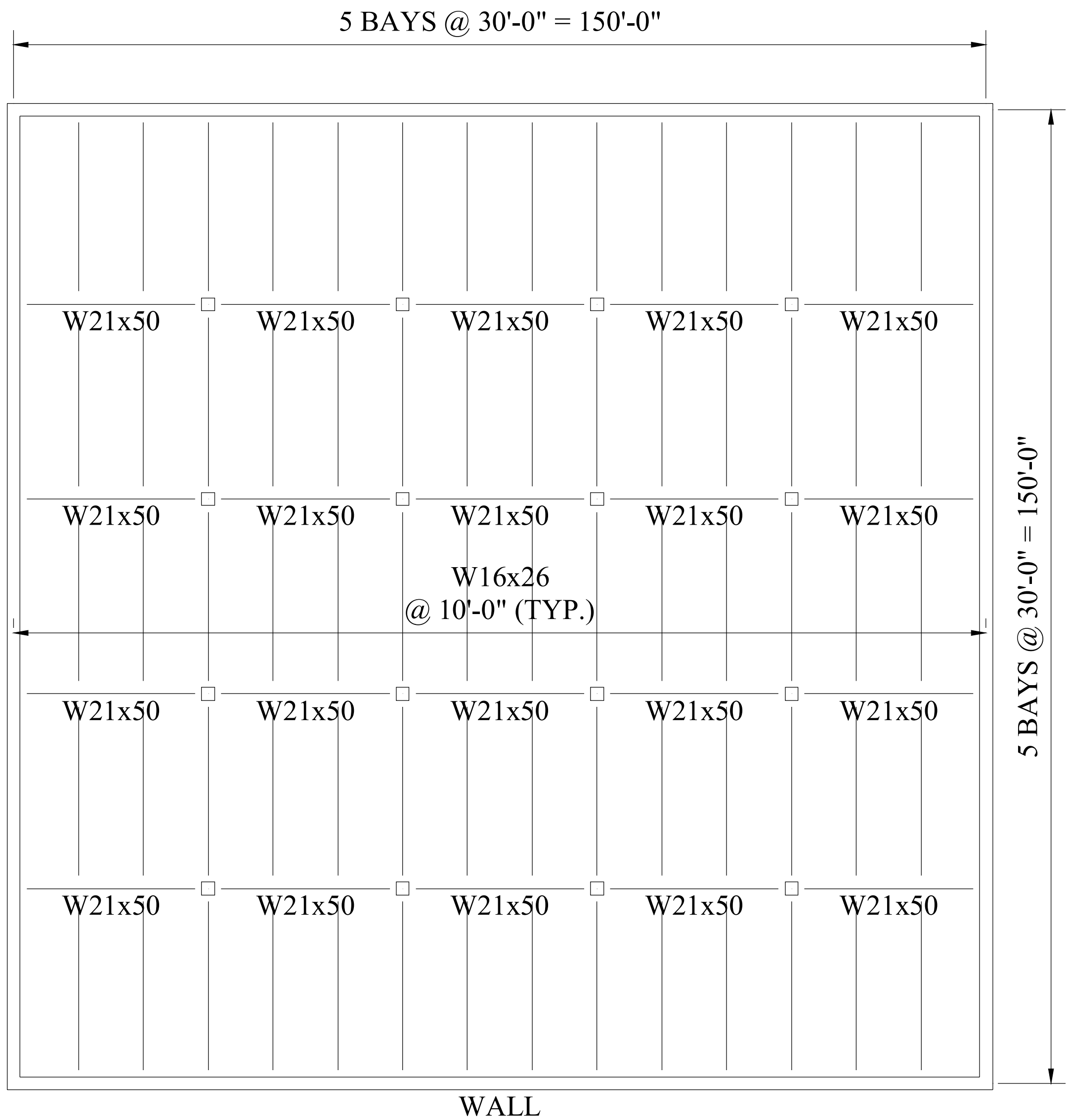


Figure 3.4 – Floor D Detailed Floor Plan

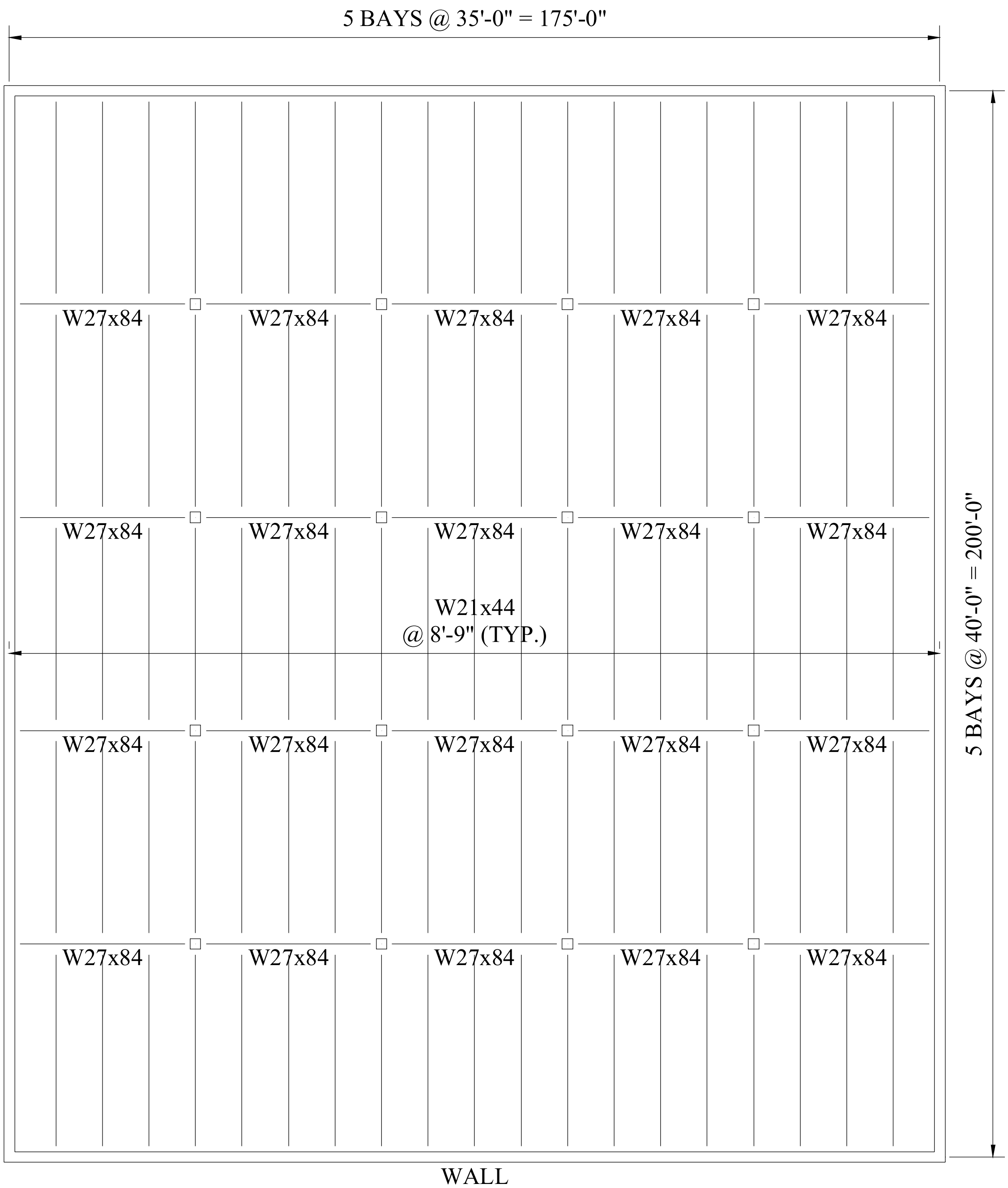


Figure 3.5 – Floor E Detailed Floor Plan

3.2.2 Different Model Set-ups

For this part of the study, five different model set-ups were used. The five set-ups are: (1) modeling the entire floor system (Full Floor), (2) a section of the floor that consists of the center nine bays (Center Section), (3) a section of the floor that consists of the strip of bays that is positioned along the beam axis which extends the entire length of the floor (Full Bay Strip), (4) a section of the floor that consists of a strip of the center three bays that are positioned along the beam axis (3 Bay Strip), and (5) the single center bay (Single Bay). The results for each case include the predicted fundamental frequency of the floor system, as well as, the peak acceleration from the seven different loading protocols listed in Tables 2.1 and 2.2. All of the model set-ups used the modeling procedure presented in Section 2.2.

Full Floor Model. For the full floor model, all 25 bays of the floor system were modeled in the structural analysis program, as shown in Figure 3.6. At the column locations, all translational degrees of freedom were restrained. Also, all exterior, or spandrel, beams and girders were assigned a frame property modifier of 1000 for the moment of inertia. This was to account for the effect of the exterior wall stiffening the spandrel beams.

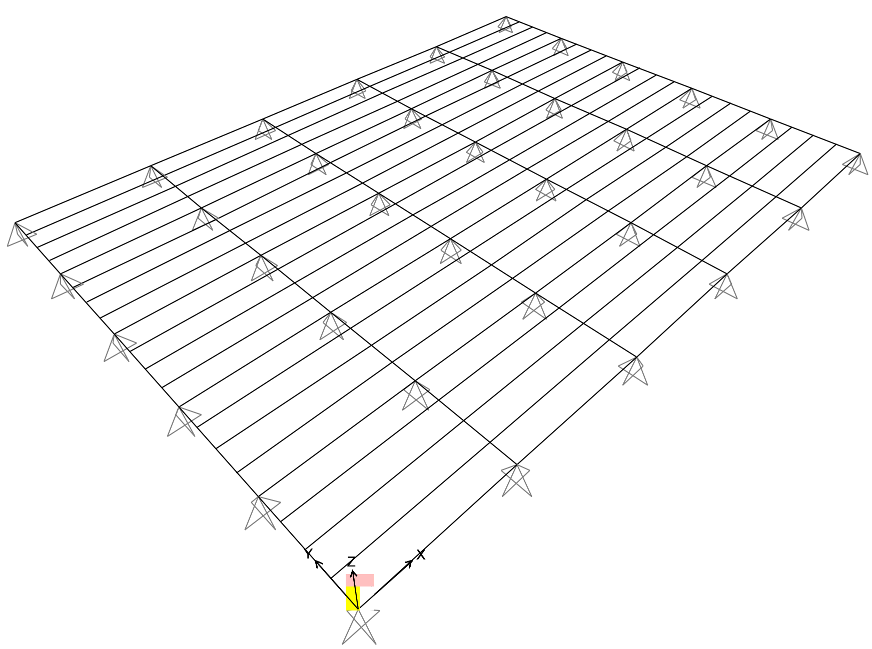


Figure 3.6 – Full Floor Model

Center Section. The center section model only includes the center nine bays, see Figure 3.7. As with the full floor model, all translational degrees of freedom at the column locations were restrained. Since there are no spandrel beams in the model, none of the frame members have a frame property modifier for the moment of inertia. However, since the exterior frame members of the model carry load from the exterior bays of the floor that are not included in the model, the exterior frame members of the model do have a frame property modifier of 0.5 for both the mass and the moment of inertia of the member. By using the frame property modifier, the exterior frame members have the same static deflection as would be expected.

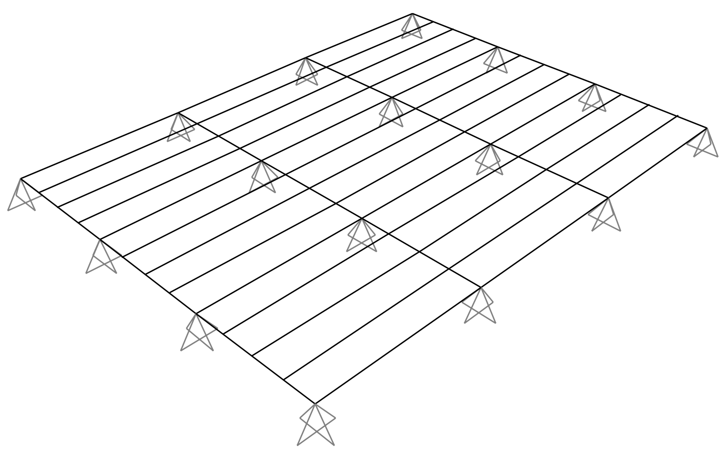


Figure 3.7 – Center Section Model

Full Bay Strip. The full bay strip model consists of all of the bays in floor that are inline with the analysis bay (along the beam axis), see Figure 3.8. As with two previous models, at the column locations the translational degrees of freedom are restrained. This model has a combination of frame property modifiers. Since the two exterior girders of the model are spandrel girders in the floor system, a frame property modifier of 1000 is assigned to the moment of inertia of the girders. One the other hand, the exterior beams in the model are actually interior beams, therefore, a frame property of 0.5 is used for the mass and moment of inertia for the same reasons as mentioned before.

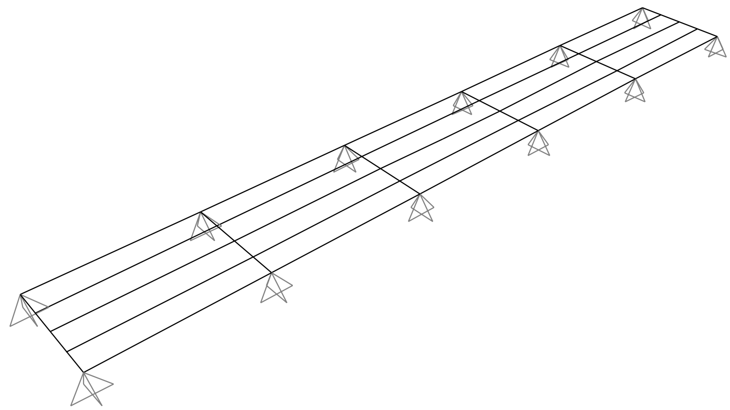


Figure 3.8 – Full Bay Strip Model

Three Bay Strip. The three bay strip model is very similar to the full bay strip model, but it only consists of the middle three bays of the strip as shown in Figure 3.9. For this model, the frame property modifiers are 0.5 for all exterior beams and girders in the model for mass and moment of inertia.

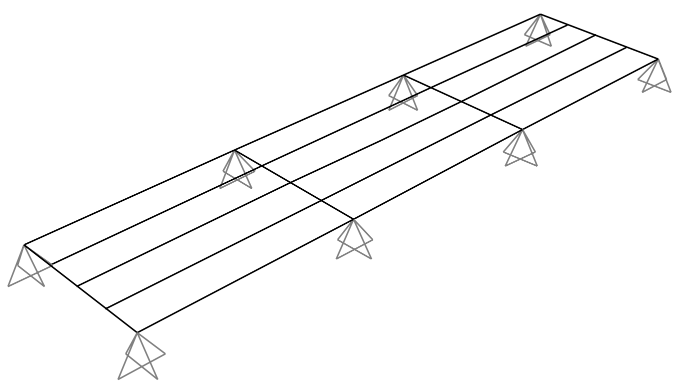


Figure 3.9 – Three Bay Strip Model

Single Bay. The single bay model only consists of the bay being analyzed for vibrations due to walking, see Figure 3.10. At each corner of the bay, all translational degrees of freedom are restrained. Also, every exterior frame element is assigned a frame property modifier of 0.5 for mass and moment of inertia.

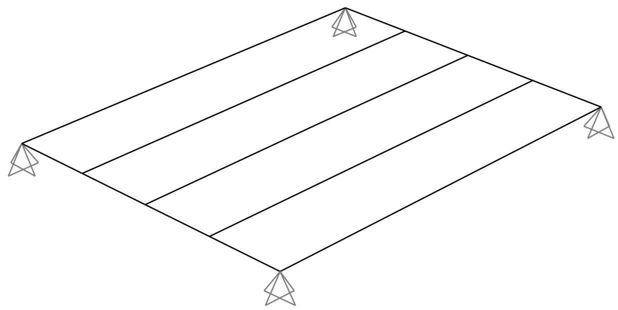


Figure 3.10 – Three Bay Strip Model

3.2.3 Predicted Fundamental Frequency Results

Table 3.3 summarizes the results from the predicted fundamental frequency comparisons from the five model analyses and the Design Guide procedure analyses. Also in the table is the ratio of the predicted fundamental frequency from the computer analysis with the predicted frequency from the Design Guide procedure. The average ratio between the Full Floor Model fundamental frequency and the Design Guide fundamental frequency (f_{FF}/f_{DG}) is 1.22 with a standard deviation of 0.13. For the Center Section Model, the average ratio between its predicted fundamental frequency and the predicted Design Guide fundamental frequency (f_{CS}/f_{DG}) is 1.17 with a standard deviation of 0.13. The Full Bay Strip Model's average ratio of the predicted fundamental frequency to the Design Guide predicted fundamental frequency (f_{FB}/f_{DG}) is 1.10 with a standard deviation of 0.03. For the Three Bay Strip Model, the average ratio of the fundamental frequency to the Design Guide fundamental frequency (f_{TB}/f_{DG}) is 1.08 with a standard deviation of 0.03. The Single Bay Model's average ratio of the predicted fundamental frequency to the Design Guide fundamental frequency (f_{SB}/f_{DG}) is 1.01 with a standard deviation of 0.02. It is noted that, except for one case, the analysis model fundamental frequency is greater than the Design Guide procedure predicted fundamental frequency. Figure 3.11 shows the results graphically. The solid line is a one-to-one correlation between the SAP2000 predicted fundamental frequency and the Design Guide procedure predicted fundamental frequency. The dashed lines are 10% lower and upper bounds.

Table 3.3 – Fundamental Frequency Results Comparisons

Floor	Design Guide	Structural Analysis Program									
		Full Floor		Center Section		Full Bay Strip		Three Bay Strip		Single Bay	
	f_{DG} (Hz)	f_{FF} (Hz)	f_{FF}/f_{DG}	f_{CS} (Hz)	f_{CS}/f_{DG}	f_{FB} (Hz)	f_{FB}/f_{DG}	f_{TB} (Hz)	f_{TB}/f_{DG}	f_{SB} (Hz)	f_{SB}/f_{DG}
Floor A	3.75	4.28	1.14	4.12	1.10	4.04	1.08	3.91	1.04	3.75	1.00
Floor B	5.25	5.83	1.11	5.62	1.07	5.59	1.07	5.55	1.06	5.33	1.02
Floor C	5.70	6.70	1.18	6.53	1.15	6.45	1.13	6.40	1.12	5.63	0.99
Floor D	4.43	5.33	1.20	5.10	1.15	4.96	1.12	4.79	1.08	4.55	1.03
Floor E	3.88	5.62	1.45	5.45	1.40	4.34	1.12	4.20	1.08	4.03	1.04
Average			1.22		1.17		1.10		1.08		1.01
Standard Deviation			0.13		0.13		0.03		0.03		0.02

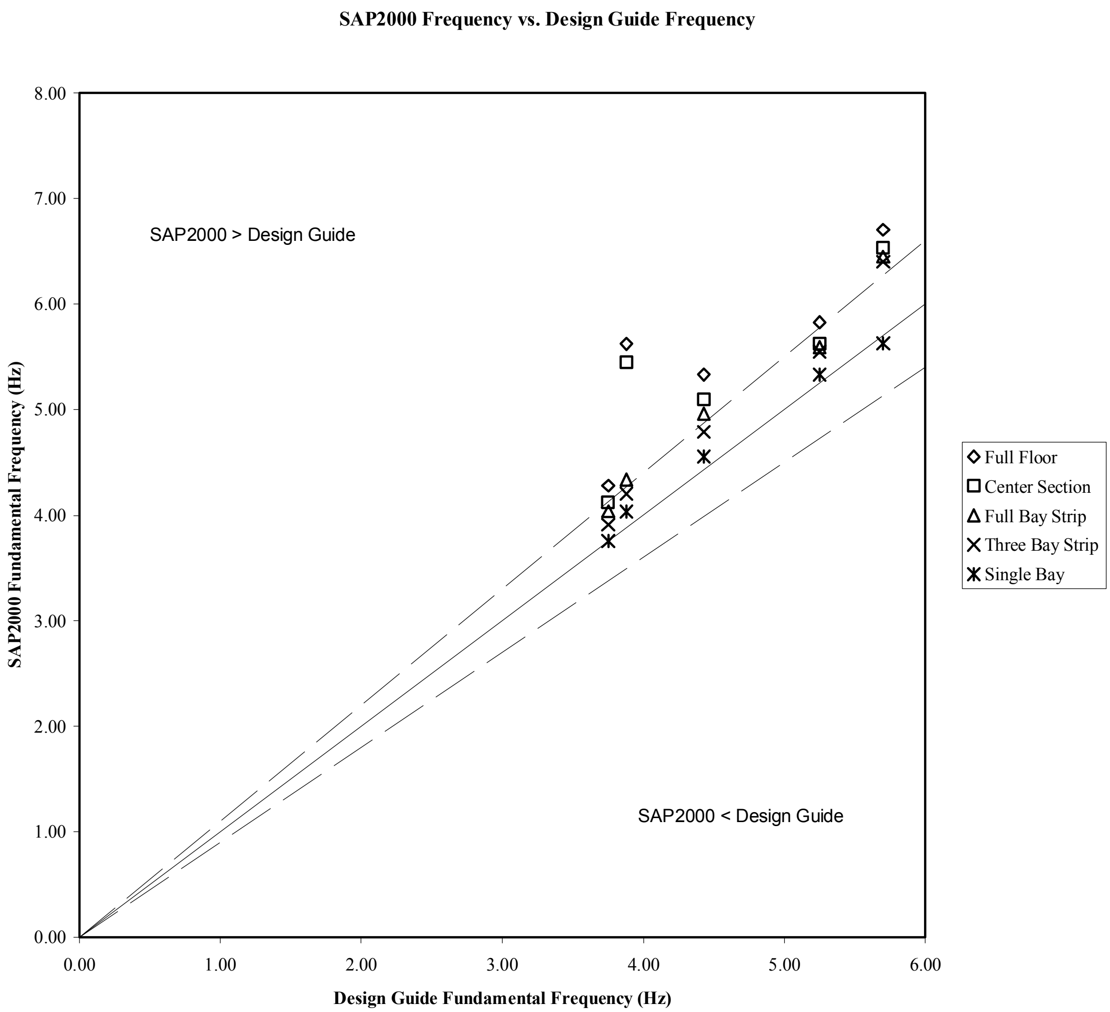


Figure 3.11 – SAP2000 vs. Design Guide Fundamental Frequencies

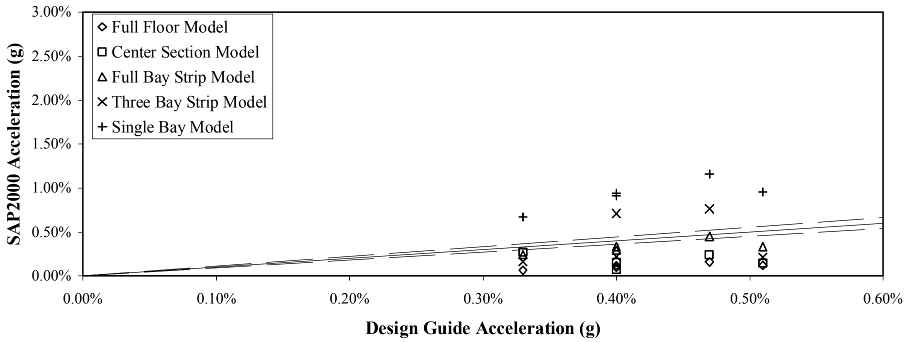
3.2.4 Predicted Peak Acceleration Results

Tables 3.4 through 3.10 summarize the predicted peak acceleration results and Figure 3.12 is a graphical representation of the data. Each table contains the predicted peak acceleration from the Design Guide procedure as well as the peak acceleration as obtained from the structural analysis program for each of the five model set-ups. Also listed in the tables is the ratio of the predicted peak acceleration from the different model set-ups to the predicted peak acceleration from the Design Guide procedure.

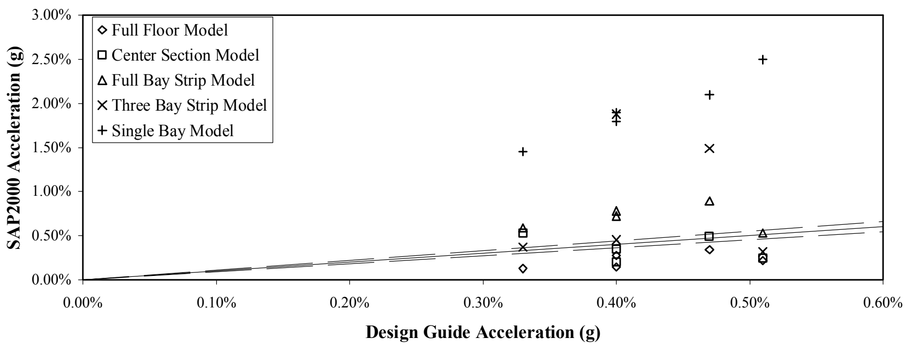
For the Design Guide Function Loading, the average ratio of the predicted peak acceleration from SAP2000 to the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model are 0.26, 0.44, 0.78, 0.97, and 2.20, respectively. The standard deviations for each of the model set-ups are 0.06, 0.25, 0.12, 0.67, and 0.24, respectively. With the exception of the Single Bay Model, the predicted accelerations from SAP2000 are less than the predicted accelerations from the Design Guide procedure. Figure 3.12(a) shows the predicted peak acceleration results for the Design Guide Function Loading. The solid line indicates a one-to-one correlation between the peak acceleration from SAP2000 to the predicted acceleration from the Design Guide procedure. The dashed lines represent 10% upper and lower bounds.

Table 3.4 – Predicted Peak Acceleration Results for Design Guide Function Loading

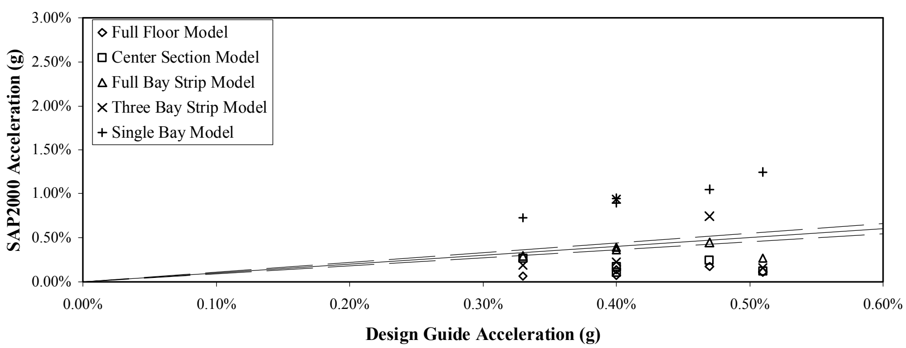
Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.11%	0.29	0.15%	0.38	0.33%	0.83	0.22%	0.54	0.91%	2.28
Floor B	0.47%	0.16%	0.34	0.24%	0.52	0.45%	0.95	0.76%	1.62	1.16%	2.47
Floor C	0.40%	0.09%	0.24	0.07%	0.18	0.29%	0.73	0.71%	1.77	0.94%	2.35
Floor D	0.51%	0.12%	0.24	0.15%	0.29	0.33%	0.65	0.21%	0.41	0.96%	1.88
Floor E	0.33%	0.06%	0.19	0.27%	0.82	0.24%	0.73	0.16%	0.49	0.67%	2.03
Average			0.26		0.44		0.78		0.97		2.20
Standard Deviation			0.06		0.25		0.12		0.67		0.24



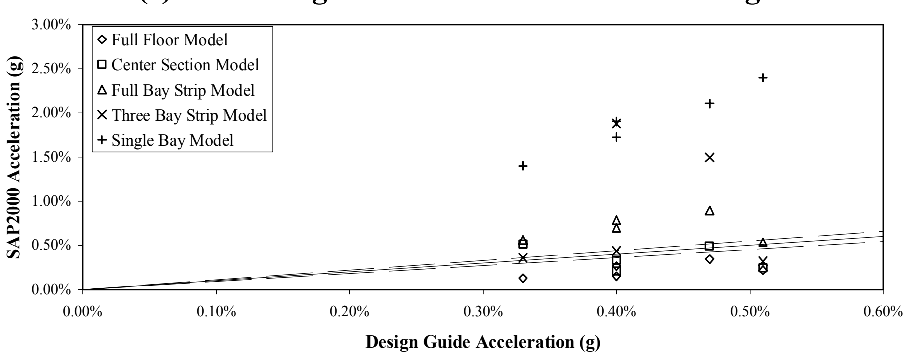
(a) Design Guide Function Loading



(b) Design Guide Fourier Series Loading

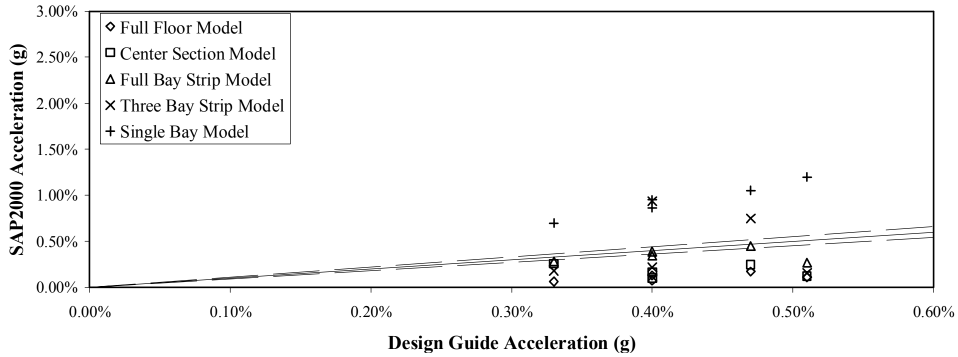


(c) 50% Design Guide Fourier Series Loading

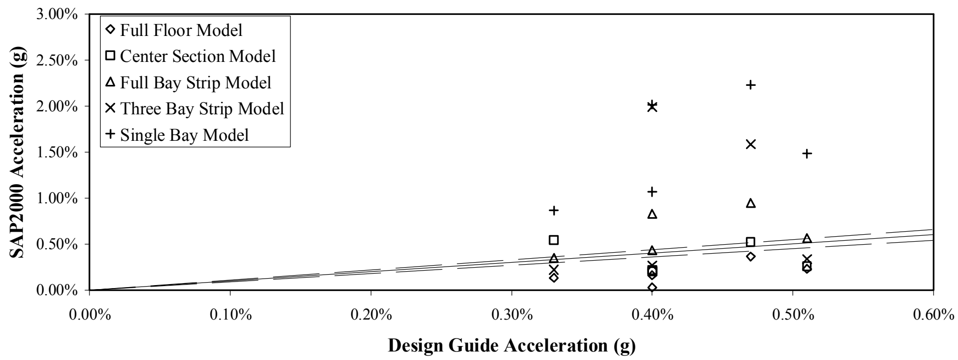


(d) Design Guide Fourier Series with Phase Shift Loading

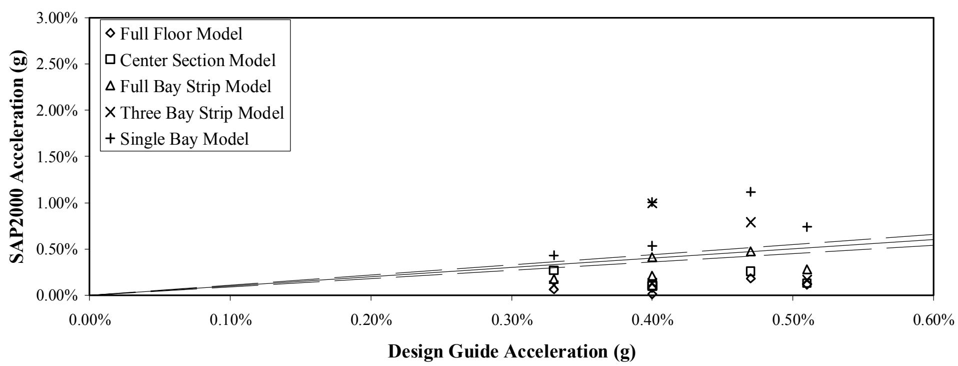
Figure 3.12 – SAP2000 Predicted Peak Acceleration vs. Design Guide Predicted Peak Acceleration



(e) 50% Design Guide Fourier Series with Phase Shift Loading



(f) IBK Fourier Series Loading



(g) 50% IBK Fourier Series Loading

Figure 3.12 – SAP2000 Predicted Peak Acceleration vs. Design Guide Predicted Peak Acceleration (continued)

The Design Guide Fourier Series Loading produces an average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model of 0.52, 0.90, 1.70, 2.15, and 4.60, respectively. The standard deviations for each of the model set-ups are 0.17, 0.46, 0.37, 1.73, and 0.21, respectively. The Full Floor Model predicts peak accelerations that are less than the Design Guide predicted acceleration and the Full Bay Strip Model and Single Bay Model predict peak accelerations that are greater than the Design accelerations. The Center Section Model and Three Bay Strip Model predict peak accelerations that are both greater and less than the Design Guide procedure predicted peak accelerations. Figure 3.12(b) is a graphical representation of the results.

Table 3.5 – Predicted Peak Acceleration Results for Design Guide Fourier Series Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.27%	0.68	0.34%	0.86	0.73%	1.81	0.45%	1.13	1.79%	4.48
Floor B	0.47%	0.34%	0.73	0.49%	1.04	0.89%	1.90	1.49%	3.17	2.10%	4.47
Floor C	0.40%	0.14%	0.36	0.20%	0.51	0.78%	1.95	1.88%	4.69	1.90%	4.74
Floor D	0.51%	0.22%	0.43	0.24%	0.48	0.53%	1.04	0.32%	0.62	2.50%	4.90
Floor E	0.33%	0.13%	0.38	0.53%	1.61	0.59%	1.78	0.37%	1.12	1.46%	4.41
Average			0.52		0.90		1.70		2.15		4.60
Standard Deviation			0.17		0.46		0.37		1.73		0.21

Using the 50% Design Guide Fourier Series Loading protocol results in an average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model of 0.26, 0.45, 0.85, 1.07, and 2.30, respectively. The standard deviations for each of the model set-ups are 0.09, 0.23, 0.19, 0.86, and 0.10, respectively. The Full Floor Model, Center Section Model, and the Full Bay Strip Model predict the peak accelerations to be less than the peak accelerations from the Design Guide procedure. The Single Bay Model predicts the peak accelerations to be greater than those from the Design Guide procedure. The Three Bay Strip Model predicts the peak accelerations to be both greater than and less than those from the Design Guide procedure. Figure 3.12(c) presents the results graphically.

Table 3.6 – Predicted Peak Acceleration Results for 50% Design Guide Fourier Series Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.14%	0.34	0.17%	0.43	0.36%	0.91	0.23%	0.56	0.90%	2.24
Floor B	0.47%	0.17%	0.36	0.25%	0.52	0.45%	0.95	0.74%	1.58	1.05%	2.23
Floor C	0.40%	0.07%	0.18	0.10%	0.25	0.39%	0.97	0.94%	2.35	0.95%	2.37
Floor D	0.51%	0.11%	0.21	0.12%	0.24	0.27%	0.52	0.16%	0.31	1.25%	2.45
Floor E	0.33%	0.06%	0.19	0.27%	0.80	0.29%	0.89	0.18%	0.56	0.73%	2.21
Average			0.26		0.45		0.85		1.07		2.30
Standard Deviation			0.09		0.23		0.19		0.86		0.10

The Design Guide Fourier Series with Phase Shift Loading protocol produces an average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model of 0.51, 0.88, 1.67, 2.13, and 4.50, respectively. The standard deviations for each of the model set-ups are 0.17, 0.44, 0.37, 1.74, and 0.23, respectively. For this loading, only the Full Floor Model predicts the peak acceleration to be consistently less than the Design Guide procedure predicted accelerations. The Full Bay Strip Model, Three Bay Strip Model, and the Single Bay model predict the peak accelerations to be greater than those predicted by the Design Guide procedure. The Center Section Model predicts peak accelerations that are greater than and less than the predicted peak accelerations from the Design Guide procedure. Figure 3.12(d) is a graphical representation of the results.

Table 3.7 – Predicted Peak Acceleration Results for Design Guide Fourier Series with Phase Shift Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.26%	0.66	0.33%	0.83	0.70%	1.75	0.43%	1.09	1.73%	4.32
Floor B	0.47%	0.34%	0.73	0.49%	1.05	0.90%	1.91	1.50%	3.18	2.11%	4.49
Floor C	0.40%	0.15%	0.37	0.20%	0.51	0.78%	1.96	1.88%	4.69	1.90%	4.76
Floor D	0.51%	0.22%	0.43	0.25%	0.48	0.53%	1.05	0.32%	0.62	2.40%	4.70
Floor E	0.33%	0.13%	0.39	0.51%	1.55	0.56%	1.71	0.36%	1.08	1.40%	4.25
Average			0.51		0.88		1.67		2.13		4.50
Standard Deviation			0.17		0.44		0.37		1.74		0.23

For the 50% Design Guide Fourier Series with Phase Shift Loading protocol, the average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model are 0.26, 0.44, 0.84, 1.07, and 2.25, respectively. The standard deviations for each of the model set-ups are 0.08, 0.22, 0.18, 0.87, and 0.11, respectively. The Full Floor Model, Center Section Model, and Full Bay Strip Model predict peak accelerations that are greater than the predicted peak accelerations from the Design Guide procedure. However, the Three Bay Strip Model predicts peak accelerations that are greater than and less than those from the Design Guide. The Single Bay Model predicts accelerations that are greater than those from the Design Guide procedure. Figure 3.12(e) shows the results graphically.

Table 3.8 – Predicted Peak Acceleration Results for 50% Design Guide Fourier Series with Phase Shift Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.13%	0.33	0.17%	0.41	0.35%	0.87	0.22%	0.54	0.86%	2.16
Floor B	0.47%	0.17%	0.37	0.25%	0.52	0.45%	0.95	0.75%	1.59	1.05%	2.24
Floor C	0.40%	0.07%	0.18	0.10%	0.25	0.39%	0.98	0.94%	2.35	0.95%	2.38
Floor D	0.51%	0.11%	0.21	0.12%	0.24	0.27%	0.52	0.16%	0.31	1.20%	2.35
Floor E	0.33%	0.06%	0.19	0.26%	0.77	0.28%	0.85	0.18%	0.54	0.70%	2.12
Average			0.26		0.44		0.84		1.07		2.25
Standard Deviation			0.08		0.22		0.18		0.87		0.11

Using the IBK Fourier Series Loading protocol results an average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model of 0.42, 0.86, 1.47, 2.07, and 3.60, respectively. The standard deviations for each of the model set-ups are 0.25, 0.51, 0.53, 2.00, and 1.19, respectively. For this loading, the Full Floor Model is the only model that consistently predicts peak accelerations less that those from the Design Guide procedure. Also, the Full Bay Strip Model and Single Bay Model are the only two that consistently predicts the peak acceleration to be higher than those from the Design Guide procedure. The other two models, the Center Section Model and the Three Bay Strip Model, predict peak accelerations that are greater than and less than those from the Design Guide procedure. Figure 3.12(f) shows these results graphically.

Table 3.9 – Predicted Peak Acceleration Results for IBK Fourier Series Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.16%	0.40	0.20%	0.51	0.43%	1.08	0.27%	0.67	1.07%	2.67
Floor B	0.47%	0.36%	0.77	0.52%	1.11	0.95%	2.02	1.58%	3.37	2.23%	4.75
Floor C	0.40%	0.03%	0.07	0.22%	0.54	0.83%	2.07	1.99%	4.97	2.01%	5.04
Floor D	0.51%	0.23%	0.45	0.26%	0.51	0.56%	1.11	0.34%	0.66	1.48%	2.91
Floor E	0.33%	0.13%	0.41	0.54%	1.64	0.35%	1.06	0.22%	0.67	0.87%	2.62
Average			0.42		0.86		1.47		2.07		3.60
Standard Deviation			0.25		0.51		0.53		2.00		1.19

The 50% IBK Fourier Series with Phase Shift Loading protocol produces an average ratio of the predicted peak acceleration from SAP2000 to the ratio of the predicted peak acceleration from the Design Guide procedure for the Full Floor Model, the Center Section Model, The Full Bay Strip Model, the Three Bay Strip Model, and the Single Bay Model of 0.21, 0.43, 0.73, 1.03, and 1.80, respectively. The standard deviations for each of the model set-ups are 0.12, 0.25, 0.26, 1.00, and 0.60, respectively. The Full Floor Model and the Center Section Model both predict peak accelerations that are less than those from the Design Guide procedure. The Full Bay Strip Model and the Three Bay Strip Model, with the exception of Floors B and C, predict the peak accelerations to be less than the Design Guide predicted peak accelerations. The Single Bay Model is the only model with the 50% IBK Fourier Series Loading that consistently predicts peak accelerations greater than the Design Guide procedure. Figure 3.12(g) presents the results in a graphical format.

Table 3.10 – Predicted Peak Acceleration Results for 50% IBK Fourier Series Loading

Floor	Design Guide	Structural Analysis Program									
		Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	a_p/g	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio	a_p/g	Ratio
Floor A	0.40%	0.08%	0.20	0.10%	0.25	0.22%	0.54	0.13%	0.34	0.53%	1.33
Floor B	0.47%	0.18%	0.39	0.26%	0.56	0.47%	1.01	0.79%	1.68	1.12%	2.37
Floor C	0.40%	0.01%	0.03	0.11%	0.27	0.41%	1.04	0.99%	2.48	1.01%	2.52
Floor D	0.51%	0.12%	0.23	0.13%	0.25	0.28%	0.55	0.17%	0.33	0.74%	1.45
Floor E	0.33%	0.07%	0.20	0.27%	0.82	0.17%	0.53	0.11%	0.33	0.43%	1.31
Average			0.21		0.43		0.73		1.03		1.80
Standard Deviation			0.12		0.25		0.26		1.00		0.60

3.2.5 Summary

In general, all five modeling set-ups in the structural analysis program predict the fundamental frequency higher than the Design Guide method. However, Boice (2003) showed that the Design Guide method has a tendency to under predict the fundamental frequency when compared to the measured fundamental frequency. The Full Bay Strip, Three Bay Strip, and Single Bay models are the most consistent in predicting the frequency at a certain percentage higher than the Design Guide method fundamental frequency. The Single Bay model tends to match the predicted fundamental frequency from the Design Guide procedure. Table 3.11 contains a summary of the data for the average ratios between the SAP2000 fundamental frequency to the Design Guide procedure predicted fundamental frequency.

Table 3.11 – Summary of Predicted Fundamental Frequency Results

Model Type	Average Ratio	Standard Deviation
Full Floor	1.22	0.13
Center Section	1.17	0.13
Full Bay Strip	1.10	0.03
Three Bay Strip	1.08	0.03
Single Bay	1.01	0.02

Table 3.12 contains a summary of the average predicted peak acceleration ratios between the predicted peak acceleration from SAP2000 and from the Design Guide procedure. From the table, the best loading protocol and model set-up combination to predict the fundamental frequency and peak acceleration for a floor system seems to be the Design Guide Function Loading protocol and the Full Bay Strip Model. This combination gives the closest average ratio of the structural analysis program peak acceleration to the predicted Design Guide peak acceleration (0.78) with one of the lowest standard deviations (0.12). Henceforth in this study, this is combination will be used to analyze floor systems.

Table 3.12 – Summary of Predicted Peak Acceleration Results

Loading Protocol	Structural Analysis Program									
	Full Floor Model		Center Section Model		Full Bay Strip Model		Three Bay Strip Model		Single Bay Model	
	Average Ratio	Standard Deviation	Average Ratio	Standard Deviation	Average Ratio	Standard Deviation	Average Ratio	Standard Deviation	Average Ratio	Standard Deviation
D.G. Function	0.26	0.06	0.44	0.25	0.78	0.12	0.97	0.67	2.20	0.24
D.G. F.S.	0.52	0.17	0.90	0.46	1.70	0.37	2.15	1.73	4.60	0.21
50% D.G. F.S.	0.26	0.09	0.45	0.23	0.85	0.19	1.07	0.86	2.30	0.10
D.G. F.S. with P.S.	0.51	0.17	0.88	0.44	1.67	0.37	2.13	1.74	4.50	0.23
50% D.G. F.S. with P.S.	0.26	0.08	0.44	0.22	0.84	0.18	1.07	0.87	2.25	0.11
IBK F.S.	0.42	0.25	0.86	0.51	1.47	0.53	2.07	2.00	3.60	1.19
50% IBK Series	0.21	0.12	0.43	0.25	0.73	0.26	1.03	1.00	1.80	0.60

Note: F.S. means Fourier Series and P.S. means Phase Shift.

3.3 CASE STUDY BUILDINGS

3.3.1 Case Study Buildings Description

This section examines the ability of using the procedure that consists of the Full Bay Strip Model and the Design Guide Loading Function protocol to predict the fundamental frequency and the predicted peak acceleration for 16 floor systems in 11 buildings located throughout the United States. All 16 of the floor systems consist of hot-rolled sections for the girders and beams with a cast in place concrete deck. For the floor systems, the measurement team made measurements from which the fundamental frequency could be determined. Also, observations to determine a subjective analysis for each floor system were recorded.

The publication by Boice and Murray (2003) contains complete data for the case studies analyzed in this study. The floor systems properties, including damping, framing, loading, slab properties and measured fundamental frequencies are included in the data. In addition, this publication contains framing plans for the floor systems analyzed.

3.3.2 Frequency Results

Table 3.13 shows the results of the predicted fundamental frequencies from the Design Guide method and the structural analysis program method. Included in the table is a comparison of the two predicted fundamental frequencies to the measured fundamental frequency (f_{DG}/f_m and f_{SAP}/f_m) as well as a comparison of the two predicted fundamental frequencies (f_{SAP}/f_{DG}). The Design Guide procedure, on average, predicts a fundamental frequency that is 82.6% of the measured fundamental frequency. The standard deviation is 11.5%. On average the SAP2000 analysis program predicts a fundamental frequency that is 92.8% of the measured frequency with a 14.7% deviation. When the two predicted fundamental frequencies are compared, the SAP2000 analysis program frequency, on average, is 112.4% of the Design Guide procedure frequencies with a standard deviation of 9.1%.

Table 3.13 – Case Study Fundamental Frequency Results Summary

Building	Bay	β (%)	Measured	Design Guide		SAP2000		Ratio
			f_m (Hz)	f_{DG} (Hz)	f_{DG}/f_m	f_{SAP} (Hz)	f_{SAP}/f_m	f_{SAP}/f_{DG}
The Jersey Lil Club	Typ.	3.0	4.93	3.15	0.639	3.239	0.657	1.028
ISU	1	2.5	3.88	3.19	0.822	3.427	0.883	1.074
	2,3	2.5	3.75	3.82	1.019	3.946	1.052	1.033
	4	2.5	4.00	3.82	0.955	3.946	0.986	1.033
Newgen Building	Ext.	3.0	6.50	4.60	0.708	5.114	0.787	1.112
Oracle Building	1,2	2.5	5.25	3.81	0.726	4.305	0.820	1.130
	3,4	2.5	4.13	3.81	0.923	4.305	1.042	1.130
	5	2.5	4.50	3.88	0.862	4.519	1.004	1.165
Newport V	Typ.	2.5	4.00	3.38	0.845	4.686	1.171	1.386
Doylestown Hosp.	Typ.	3.0	7.00	4.69	0.670	5.607	0.801	1.195
SDL	Typ.	3.0	N/A	3.29	N/A	3.561	N/A	1.082
AMEX	Typ.	2.5	5.25	4.24	0.808	4.384	0.835	1.034
GTE	Typ.	3.0	5.50	4.00	0.727	4.376	0.796	1.094
Three Parkwood	1	3.0	4.25	4.20	0.988	4.918	1.157	1.171
Norman Pointe 1	1	3.0	5.00	4.35	0.870	4.935	0.987	1.135
	4	3.0	5.25	4.35	0.829	4.935	0.940	1.135
Average					0.826		0.928	1.124
Standard Deviation					0.115		0.147	0.091

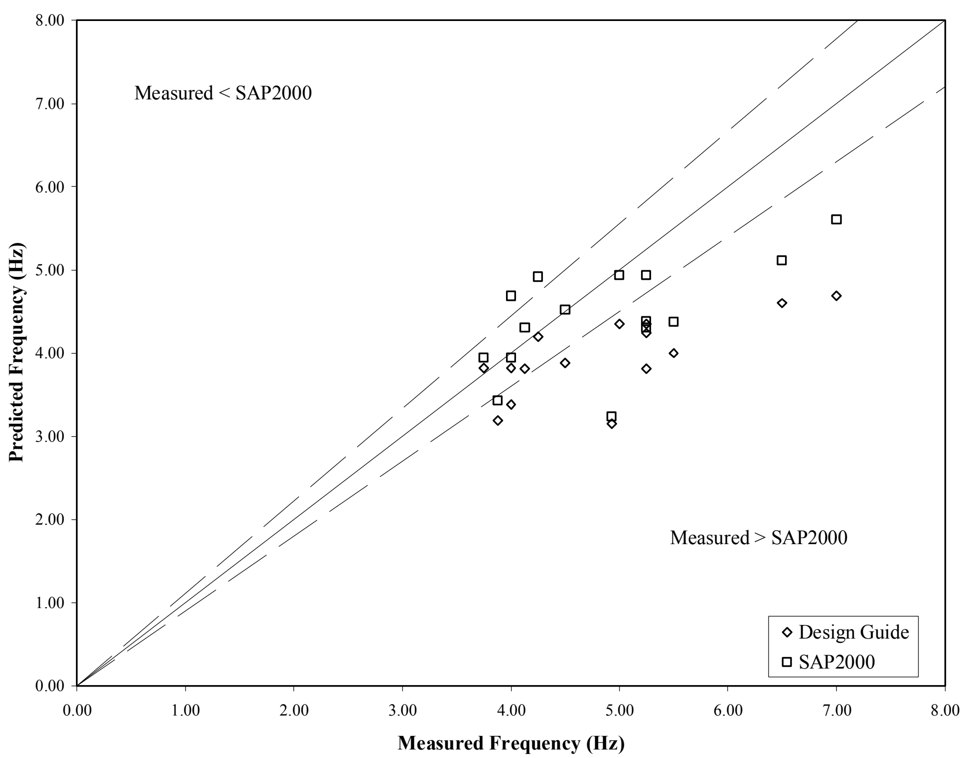


Figure 3.13 – Predicted Frequency vs. Measured Frequency

3.3.3 Predicted Acceleration Results

Table 3.14 contains the predicted peak accelerations from the two sets of analyses: the Design Guide procedure and the structural analysis program procedure. The table also contains the observed subjective evaluations. When the predicted peak acceleration from the structural analysis program is compared to the predicted peak acceleration from the Design Guide, the structural analysis program result is, on average, 87.3% of the Design Guide procedure. However, the standard deviation is 54.6%. As can be seen in Figure 3.14, the accelerations from the structural analysis program analyses are scattered when compared to the Design Guide predicted accelerations. The agreement column in Table 3.14 is a comparison of the SAP2000 results and the acceptable limit of 0.50%g to the subjective evaluations. The SAP2000 results agree with the subjective evaluation in only four of the cases.

Table 3.14 – Case Study Predicted Peak Acceleration Results Summary

Building	Bay	β (%)	Subjective Analysis	D. G.		SAP2000		Ratio
				a_p/g	Agreement	a_p/g	Agreement	a_{SAP}/a_{DG}
The Jersey Lil Club	Typ.	3.0	NC	0.34%	Y	0.54%	N	1.578
ISU	1	2.5	MC	0.95%	Y	0.48%	N	0.501
	2,3	2.5	MC	1.02%	Y	0.43%	N	0.424
	4	2.5	MC	1.01%	Y	0.17%	N	0.172
Newgen Building	Ext.	3.0	NC	0.68%	N	0.41%	Y	0.599
Oracle Building	1,2	2.5	C	0.80%	Y	0.42%	N	0.523
	3,4	2.5	C	0.80%	Y	0.42%	N	0.523
	5	2.5	C	0.94%	Y	0.42%	N	0.446
Newport V	Typ.	2.5	NC	0.46%	Y	0.46%	Y	1.002
Doylestown Hosp.	Typ.	3.0	NC	0.31%	Y	0.30%	Y	0.953
SDL	Typ.	3.0	C	0.40%	N	0.35%	N	0.876
AMEX	Typ.	2.5	C	0.56%	Y	1.14%	Y	2.043
GTE	Typ.	3.0	MC	0.43%	N	0.26%	N	0.607
Three Parkwood	1	3.0	MC	0.59%	Y	0.40%	N	0.680
Norman Pointe 1	1	3.0	NC	0.38%	Y	0.58%	N	1.520
	4	3.0	NC	0.38%	Y	0.58%	N	1.520
Average								0.873
Standard Deviation								0.546

Notes:

1. "NC" represents no complaints, "C" represents complaints, and "MC" represents many complaints.
2. "Y" represents agreement and "N" disagreement between the subjective analysis and the prediction.

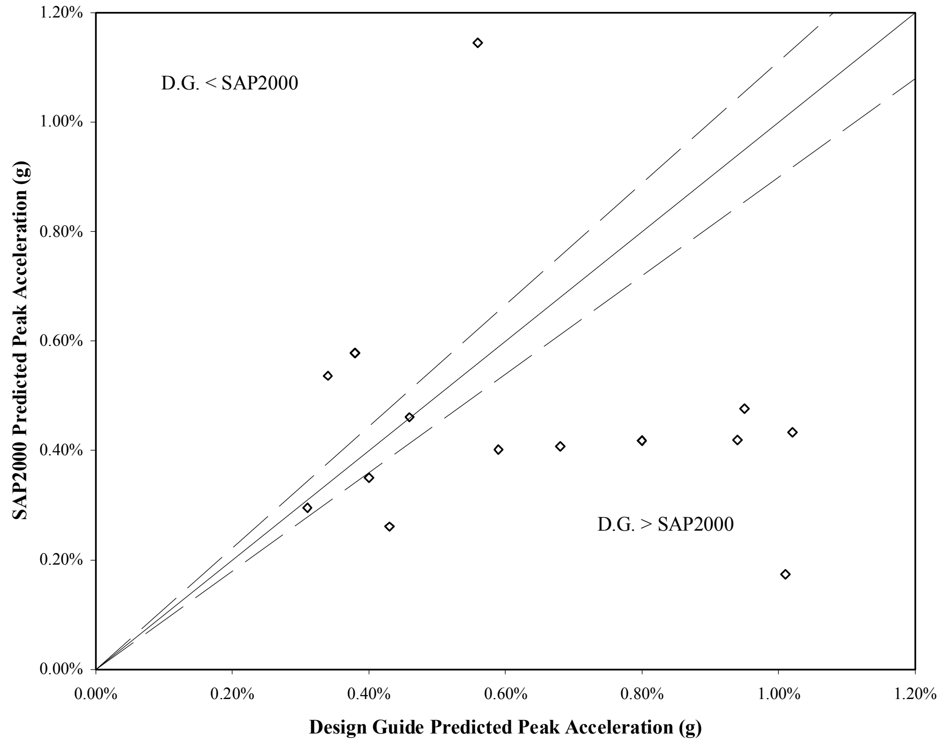


Figure 3.14 – Structural Analysis Program vs. Design Guide Predicted Acceleration

3.3.4 Summary

The main goal of this part of the study was to determine if the procedure to use a commercially available structural analysis program provided results that agree with the Design Guide results for a number of existing buildings. The structural analysis program procedure results in a predicted fundamental frequency that is more accurate than the Design Guide procedure, which can be seen in Table 3.13 and Figure 3.13. On the other hand, the Design Guide procedure does a much better job at predicting the acceptability of a floor system when compared to subjective analyses than the structural analysis program procedure. This can be seen in Table 3.13.

CHAPTER IV
ANALYTICAL CASES AND RESULTS

4.1 INTRODUCTION

In the previous chapter, it was shown that using a structural analysis program to model a floor system adequately predicts the fundamental frequency of the system, but does not predict a peak acceleration that matches the Design Guide procedure. The purpose of this chapter is to study the Design Guide, structural analysis procedure, and additional analytical procedures for multiple cases so that sources of differences between the methods might be identified. The first section of this chapter is a derivation of the effective panel width equations used in the Design Guide method. Since the Design Guide method uses different effective panel widths than the other methods, the Design Guide approach must be understood. Three types of systems: one-way systems, two-way symmetric systems, and two-way asymmetric systems, are analyzed under a dynamic loading in this chapter using the Design Guide Method, the Rayleigh Method, the Analytical Method, and the structural analysis program method to obtain the fundamental frequency and peak acceleration of the system. The results from these analyses are then compared to see where sources of differences between the Design Guide method and the structural analysis program method may exist.

4.2 DERIVATION OF THE DESIGN GUIDE EFFECTIVE PANEL WIDTH EQUATION

Allen and Murray (1993) state that the effective panel width relationship presented in the Design Guide is derived from orthotropic plate theory, but no derivation is shown. Since orthotropic plate theory can be related to isotropic plate theory, and to simplify the equations, isotropic plate vibration relationships are used to develop the derivation. First, the effective mass of an isotropic plate is to be determined. IBK (1995) lists several coefficients to convert the uniformly distribute mass of a plate to a lumped mass so the plate may be considered as a single degree of freedom (SDOF) system. These coefficients vary linearly with the aspect ratio of the plate. The aspect ratio, η, is defined as the ratio of the width-to-length of the plate. Therefore, the effective mass of a plate for a SDOF vibration problem can be expressed as:

$$M_{eff} = \alpha\, M_{PL} \tag{4.1}$$

where M_{eff} is the effective mass, M_{PL} is the total mass of the plate, and α is the coefficient to convert the total mass to the effective mass. The coefficient α, since it is a linear function, can be expressed as a function of the aspect ratio.

$$\alpha = P\eta + Q \tag{4.2}$$

Here P and Q are arbitrary constants. If the length of the plate is maintained and the only variable that changes is the width, Equation 4.1 can be rewritten as:

$$a_{eff}b\textit{m} = \alpha ab\textit{m} \tag{4.3}$$

where a_{eff} is the effective width, m is mass per unit area of the plate, and a and b are the width and length of the plate, respectively. Combining Equations 4.3 and 4.2, Equation 4.4 is obtained.

$$a_{eff} = (P\eta + Q)a \tag{4.4}$$

Since Allen and Murray (1993) assume that the floor behaves as an orthotropic plate, a conversion from the isotropic equations must be made. Timoshenko and Woinowsky-Krieger (1959) show that to use isotropic plate equations for orthotropic plates, the width "a" needs to be replaced by a_0, from

$$a_0 = a(D_y/D_x)^{1/4} \tag{4.5}$$

where D_x and D_y are the rigidities of the plate about the x and y axis, respectively. Using Equation 4.5, the aspect ratio becomes:

$$\eta = a(D_y/D_x)^{1/4}/b \quad \textbf{(4.6)}$$

Then substituting $b\eta(D_x/D_y)^{1/4}$ for a, Equation 4.7 is obtained.

$$a_{eff} = (P\eta + Q)\eta b(D_x/D_y)^{1/4} \quad \textbf{(4.7)}$$

If the quantity $P\eta^2 + Q\eta$ is assumed to be constant, then Equation 4.8 is obtained.

$$a_{eff} = Rb(D_x/D_y)^{1/4} \quad \textbf{(4.8)}$$

where R is an arbitrary constant. Converting the above variables to those used in the Design Guide, a_{eff} is B_j, D_x is D_s, D_y is D_j, R is C_j, and b is L_j, the following equation is obtained.

$$B_j = C_j(D_s/D_j)^{1/4}L_j \quad \textbf{(4.9)}$$

Allen and Murray (1993) state that C_j was calibrated to existing floors to be either 1.0 or 2.0 depending on the type of bay being considered: 1.0 if it is a mezzanine with a beam along the free edge and 2.0 otherwise. It can be deduced that the coefficient R is C_j. They also recommend that an upper limit of $^2/_3$ of the floor width be imposed on the effective panel width. This upper limit accounts for dispersion of vibrations in the system. The same type of derivation can be made for a girder panel.

4.3 DESCRIPTION OF ANALYSIS TYPES

4.3.1 Design Guide Method and Structural Analysis Program Method

The systems in this chapter were analyzed using the Design Guide Method and Structural Analysis Method as described in Sections 1.2 and 2.2, respectively. In addition, for the one-way systems, an alternate form of the Design Guide Method was used. In this method, instead of using the effective panel width as calculated, the entire width of the system was used. This allows direct correlation of the Design Guide Analysis Method to the other methods without concern for discrepancies caused by different effective panel widths. For the two-way systems, the Design Guide Alternate Method consists of using the full width and length of the systems for the same reasons as for the one-way systems.

4.3.2 Rayleigh Method

The Rayleigh Method provides additional results which were compare with both the Design Guide method results and structural analysis program method results. The Rayleigh Method is an approximate method that can be used to obtain the effective mass of a system. The effective mass for a one-way system, m_e, is

$$m_e = \int_0^L m(x)(\phi((x))^2 dx \tag{4.10}$$

where $m(x)$ is the mass per unit length of the system (constant for floor systems), $\phi(x)$ is the assumed mode shape of the system with an amplitude of 1.0 at the point at which is of concern. The effective mass, m_e, for a two-way floor system is:

$$m_e = \int_0^{L_j}\int_0^{L_g} m(x,y)(\phi((x,y))^2 dxdy \tag{4.11}$$

where $m(x,y)$ is the mass per unit area of the system (constant for floor systems), $\phi(x,y)$ is the assumed mode shape of the system with an amplitude of 1.0 at the midpoint of the system.

The stiffness of the system is treated the same as a series of springs in combination where the spring constants are the stiffness of the beams and girders. If the beams are in parallel, e.g. two girders, their stiffnesses are added together. If the members are in series, e.g. a beam and girder, the stiffnesses are related by:

$$\frac{1}{k_s} = \frac{1}{k_1} + \frac{1}{k_2} \tag{4.12}$$

where k_s is the stiffness of the spring in series and k_1 and k_2 are the stiffnesses of the components of the system, e.g. beams and girders. The stiffness of the system, k_e, is found by using the combinations of springs in series and parallel where applicable.

Once the stiffness and mass of the system are known, the fundamental frequency is given by:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k_e}{m_e}} \tag{4.13}$$

where k_e is the effective stiffness of the system and m_e is the effective mass. Once the fundamental frequency of a system is known, the peak acceleration is:

$$\frac{a_p}{g} = \frac{p_0}{2\beta g m_e} \tag{4.14}$$

where p_0 is the amplitude of the forcing function, β is the damping ratio, m_e is the effective mass of the system and g is the acceleration due to gravity.

4.3.3 Analytical Method

The Analytical Method is used to provide an additional method's results to compare the results from the Design Guide method and the structural analysis program method. The Analytical Method involves modeling each component of the system as a mass-spring-damper SDOF system, then adding the component frequencies using Dunkerly's Equation:

$$\frac{1}{f_n^{\,2}} = \frac{1}{f_j^{\,2}} + \frac{1}{f_g^{\,2}} \tag{4.15}$$

where f_n is the fundamental frequency of the system and f_j and f_g are the fundamental frequencies of the components of the system. Figure 4.1 shows an example of the idealized system. The component fundamental frequencies are calculated using Equation 4.13. The effective mass of the system is calculated using:

$$m_e = m_j \frac{k_e}{k_j} + m_g \frac{k_e}{k_g} \tag{4.16}$$

where m_e is the effective mass of the system, m_j and m_g are the effective masses of the components of the system, k_e is the effective stiffness of the system, and k_j and k_g are the stiffnesses of the components, e.g. beams and girders. The effective stiffness of the system is calculated in the same manner as for the Rayleigh Method. Once the effective mass and fundamental frequency is known, Equation 4.14 is used to predict the peak acceleration of the system.

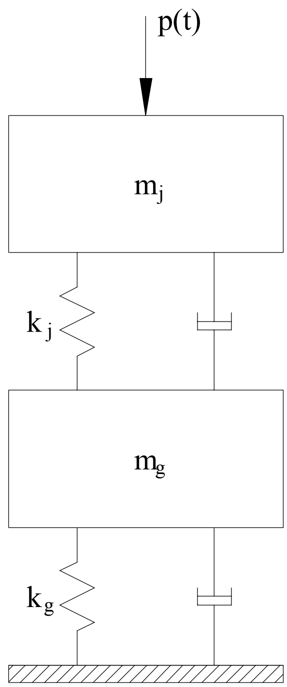


Figure 4.1 – Analytical Model Example

4.4 ANALYSIS OF ONE-WAY SYSTEMS

4.4.1 Description

One-way systems consist of only one type of flexible member. Examples include simply-supported beams (footbridges), continuous beams, floor systems with rigid girders or walls, and floor systems with rigid beams and flexible girders. Four different one-way systems were analyzed using the Design Guide method, the Alternate Design Guide Method, the structural analysis program method, the Rayleigh Method, and the Analytical Method.

System 1 is a footbridge with two hot-rolled W21x44 steel sections spanning 40 ft that support a concrete slab. Figure 4.2 shows the cross section of the system. The composite moment of inertia for the footbridge is 5,818 in^4 and the weight is 838 plf. The loading protocol is defined by Equation 4.17 and is applied at the center of the span.

$$p(t) = 9.10\cos(2\pi f_n t) \quad \textbf{(4.17)}$$

Here f_n is the fundamental frequency, t is time in seconds, and 9.10 is the amplitude of the loading function, lbs.

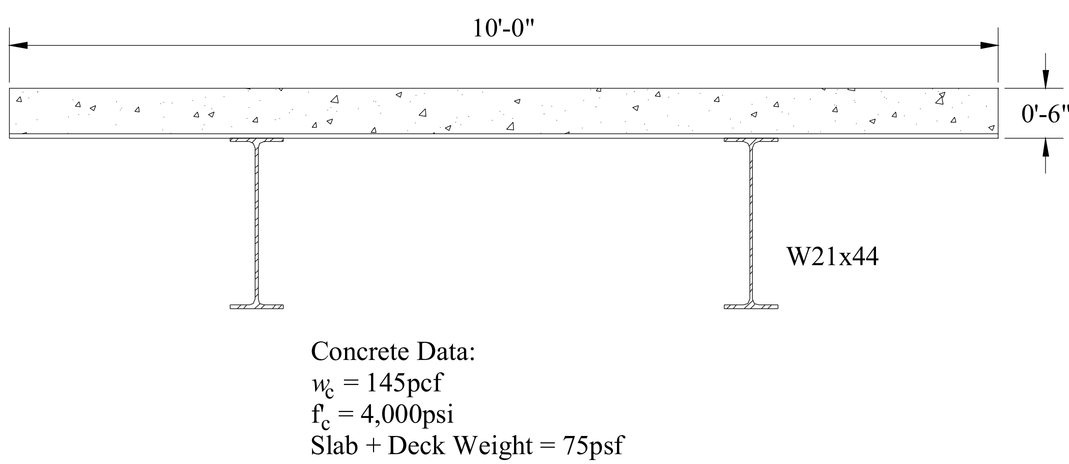


Figure 4.2 – System 1 Cross Section

System 2 is a three span, simply-supported beam system. Each span of the beam is 20 ft, the beam has an assumed moment of inertia of 1000 in^4, and the assumed beam weight is 1000 plf. Figure 4.3 shows a schematic of the system. The loading function is Equation 4.17 and is applied at the midpoint of the center span. For the Design guide procedure, instead of using an amplitude of $65e^{-0.35fn}$, 9.10 lb is used to provide consistency in the results between methods.

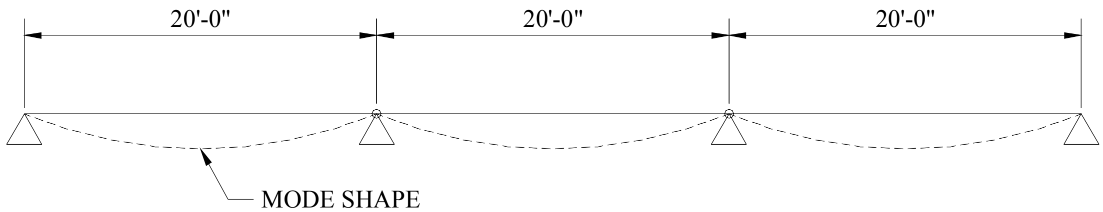


Figure 4.3 – System 2

System 3 is a single bay floor system where the beams are supported by walls. The floor is 40 ft (beam span) by 30 ft (equivalent girder span). The beams are W18x35 spaced at 6 ft on center. This system represents the beam mode vibrations found in a two-way floor system. Figure 4.4 shows the details of the system. The loading function for this system is the same as for Systems 1 and 2, Equation 4.17.

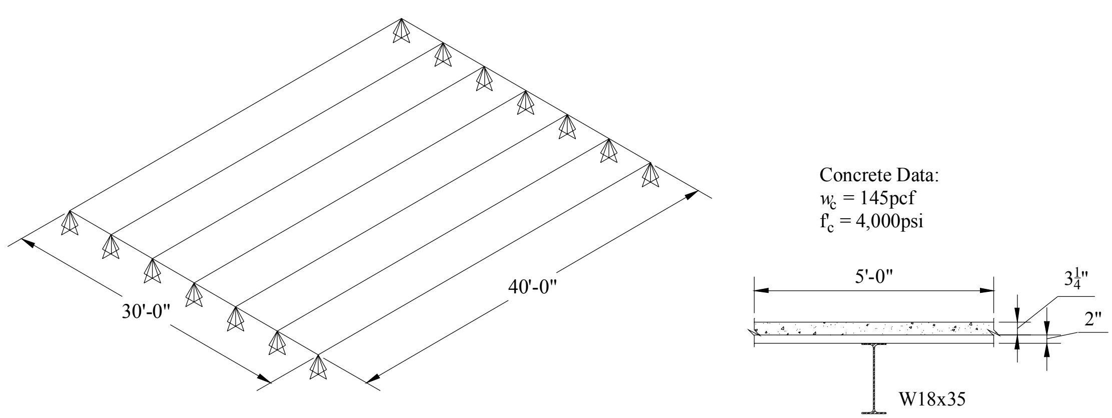


Figure 4.4 – System 3

System 4 is similar to System 3 except the beams are rigid and are supported by W24x62 girders. This system represents the girder mode of vibration in two-way systems. Figure 4.5 shows details.

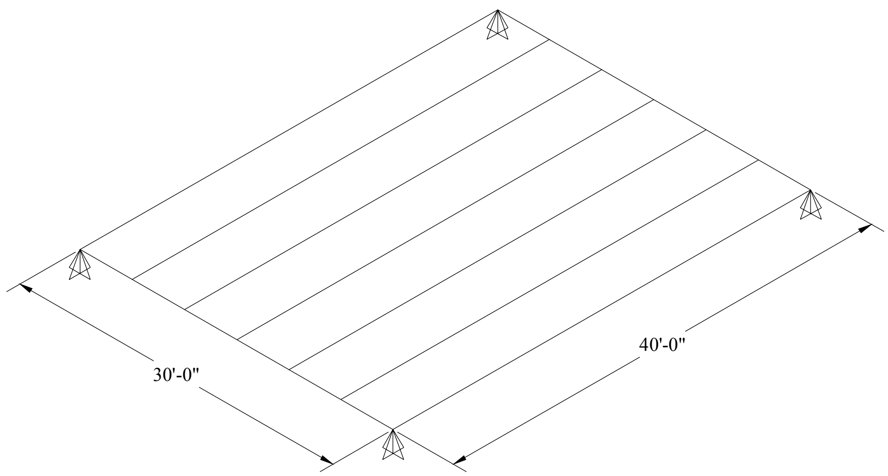


Figure 4.5 – System 4

The analysis methods used in this part of the study are the Design Guide and Design Guide Alternate Methods, the Rayleigh Method, the Analytical Method, and the structural analysis program method. The results of these analyses are the predicted fundamental frequencies and peak accelerations.

4.4.2 Results

Table 4.1 is a summary of the results of the different analyses performed on the four systems. Since the purpose of this part of the study is to try to find differences between the basic Design Guide Method and the other methods, all results are with respect to the basic Design Guide Method. For System 1, all four analysis methods predicted the frequency within 1.1% of the Design Guide Method. All three hand methods predicted the same peak acceleration and the structural analysis program predicted peak acceleration was 2.9% lower than the other three methods.

For System 2, five analysis methods were used. The Design Guide Alternate method is the Design Guide method with the continuity factor for the beams equal to 3, not 1.5 as recommended, because the system consists of three spans. All of the methods predicted the fundamental frequency as approximately 10 Hz. With the exception of the basic Design Guide method, the peak acceleration of the system was predicted to be about 1.50%g. Because of the difference in the continuity factor, the Design Guide method's predicted acceleration was double the acceleration of the other methods.

Table 4.1 – Summary of One-way System Results

System	Analysis Method	Effective Weight, lb	Frequency		Acceleration		Remarks
			f_n, Hz	% Difference	a_p/g, %	% Difference	
System 1	Design Guide	33520	6.61	-	2.71	-	B_j = 10 ft
	Rayleigh	33520	6.54	1.1%	2.71	0.0%	
	Analytical	33520	6.54	1.1%	2.71	0.0%	
	SAP2000	-	6.59	0.4%	2.63	2.9%	
System 2	Design Guide	30000	10.04	-	3.03	-	
	Design Guide Alt.	60000	10.04	0.0%	1.52	50.0%	Continuity factor = 3
	Rayleigh	60000	9.92	1.1%	1.52	50.0%	
	Analytical	60000	9.92	1.1%	1.52	50.0%	
	SAP2000	-	10.00	0.4%	1.48	51.0%	
System 3	Design Guide	103489	5.44	-	0.29	-	B_j = 34.77 ft
	Design Guide Alt.	89280	5.44	0.0%	0.34	-15.9%	B_j = 30 ft
	Rayleigh	89280	5.38	1.1%	0.34	-15.9%	
	Analytical	89280	5.38	1.1%	0.34	-15.9%	
	SAP2000	-	5.41	0.5%	0.36	-22.3%	
System 4	Design Guide	61875	7.68	-	0.49	-	B_j = 26.67 ft
	Design Guide Alt.	93012	7.68	0.0%	0.33	33.5%	B_j = 40 ft
	Rayleigh	93012	7.60	1.1%	0.33	33.5%	
	Analytical	93012	7.60	1.1%	0.33	33.5%	
	SAP2000	-	7.71	-0.4%	0.32	34.9%	

Note: Effective weight is $2m_eg$.

Five analysis were also conducted for System 3. For the Design Guide Alternate method, the effective width for the beam panel was taken as the width of the bay. All methods predict the fundamental frequency of the system as approximately 5.4 Hz. With the exception of the Design Guide method, the predicted peak acceleration was approximately 0.34%g. The Design Guide Method predicts a peak acceleration of 0.29%g.

For System 4, all of the methods predict the fundamental frequency of the system to be about 7.65 Hz. Again, with the exception of the Design Guide method, the predicted peak accelerations from the different methods were approximately equal. The Design Method's use of the calculated B_g instead of the full width of the system produced a percent difference of 33.5% in the predicted peak acceleration.

4.5 ANALYSIS OF TWO-WAY SYMMETRIC SYSTEMS

4.5.1 Description

This section examines three systems in which symmetry in the framing is present. The systems were subjected to the dynamic loading at the midpoint of the system as defined in Equation 4.17. The systems were analyzed using the Design Guide Method, the Design Guide Alternate Method, the Rayleigh Method, the Analytical Method, and the structural analysis program method. The systems are similar to Systems 3 and 4; however, the beams and girders consist of different sections and moment of inertias, see Table 4.2. Figure 4.6 shows details of the systems.

Table 4.2 – Systems 5 through 7 Framing Properties

System	Girder		Beam	
	Section	I_{comp} (in^4)	Section	I_{comp} (in^4)
System 5	W24x62	4598	W18x35	1748
System 6	W27x84	7380	W18x35	1748
System 7	W24x62	4598	W24x55	3924

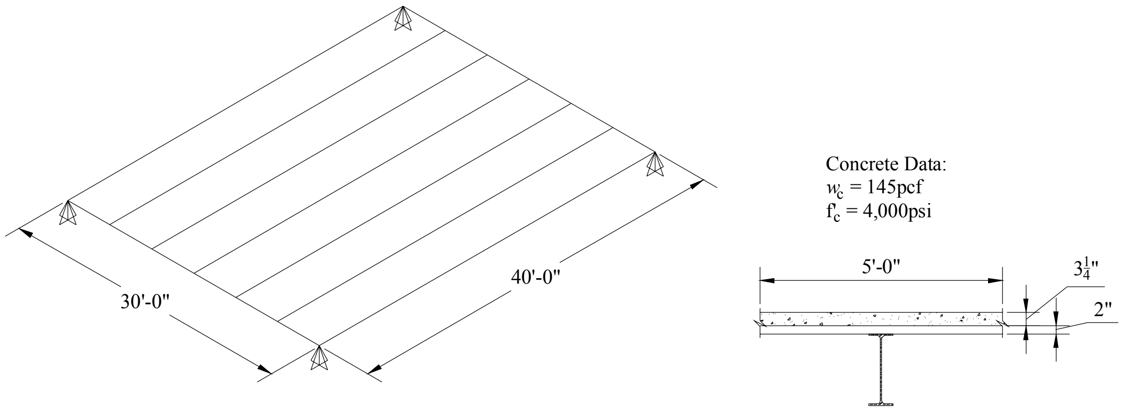


Figure 4.6 – Systems 5 through 7

4.5.2 Results

Table 4.3 summarizes the results of the different analyses performed on the two-way symmetric systems. For System 5, all five methods predicted fundamental frequencies within 2.08% of each other. The range for the predicted fundamental frequencies is 4.54 Hz to 4.64 Hz. However, the five methods predict peak accelerations that range from 0.33%g to 0.42%g with the Design Guide Method being the smallest.

System 6 has similar results as System 5. The predicted fundamental frequencies ranged from 4.82 Hz to 4.90 Hz which are within 1.7% of each other. The predicted peak accelerations ranged from 0.32%g to 0.42%g for this system with the Design Guide Method being the smallest.

The System 7 predicted fundamental frequencies were not as close to each other as in Systems 5 and 6; they ranged from 5.45 Hz to 5.85 Hz which is within 7.3% of each other. The predicted peak accelerations range from 0.35%g to 0.43%g with the Analytical Method being the smallest.

Table 4.3 – Summary of Two-way Symmetric System Results

System	Analysis Method	Effective Weight, lb	Frequency		Acceleration		Remarks
			f_n, Hz	% Difference	a_p/g (%)	% Difference	
System 5	Design Guide	90967	4.54	-	0.33	-	B_j = 34.77 ft, B_g = 26.67 ft
	Design Guide Alt.	76363	4.54	0.0%	0.40	-19.1%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	81758	4.64	-2.1%	0.37	-11.3%	
	Analytical	85457	4.64	-2.1%	0.35	-6.4%	
	SAP2000	-	4.64	-2.1%	0.42	-27.2%	
System 6	Design Guide	94776	4.82	-	0.32	-	B_j = 34.77 ft, B_g = 26.67 ft
	Design Guide Alt.	80277	4.82	0.0%	0.38	-18.1%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	83102	4.89	-1.4%	0.37	-14.0%	
	Analytical	86817	4.89	-1.4%	0.35	-9.2%	
	SAP2000	-	4.90	-1.7%	0.42	-31.4%	
System 7	Design Guide	76454	5.45	-	0.40	-	B_j = 28.41 ft, B_g = 26.67 ft
	Design Guide Alt.	70149	5.45	0.0%	0.43	-9.0%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	85199	5.85	-7.3%	0.36	10.3%	
	Analytical	87548	5.85	-7.3%	0.35	12.7%	
	SAP2000	-	5.69	-4.5%	0.43	-7.3%	

Note: Effective weight is $2m_eg$.

4.6 ANALYSIS OF TWO-WAY ASYMMETRIC SYSTEMS

4.6.1 Description

This section examines three systems in which symmetry in the framing is not present. The systems were subjected to the dynamic loading given in Equation 4.17, applied in the center of the system. The systems were analyzed using the Design Guide Method, Design Guide Alternate Method, the Rayleigh Method, the Analytical Method, and the structural analysis program method. The systems are similar to the systems examined in Section 4.2, except the girders in the model consist of different sections and moment of inertias, see Table 4.4. Figure 4.7 shows details of the system.

Table 4.4 – Systems 8 through 10 Framing Properties

System	Left Girder		Right Girder	
	Section	I_{comp} (in^4)	Section	I_{comp} (in^4)
System 8	W24x62	4598	Wall	∞
System 9	W24x62	4598	2 x Left Girder	9197
System 10	W24x62	4598	½ x Left Girder	2299

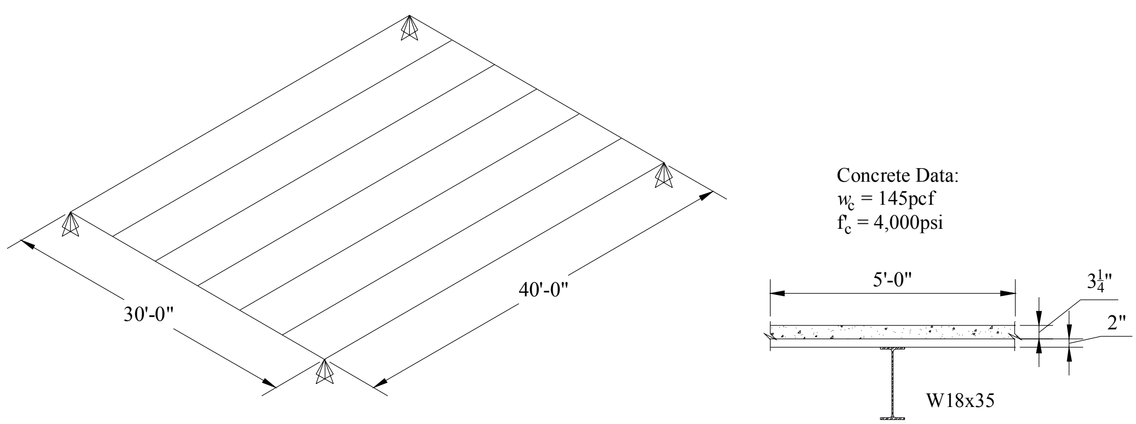


Figure 4.7 – Systems 8 through 10

4.6.2 Results

Table 4.5 summarizes the results for the two-way asymmetric systems. The different analysis types predicted the fundamental frequency of System 8 ranged from 4.52 Hz to 4.97 Hz and which are within 10% of each other. The predicted peak accelerations ranged from 0.33%g to 0.41%g with the Design Guide Method being the smallest.

The fundamental frequencies for System 10 were predicted within 12.3% of each other, with a range of 3.99 Hz to 4.80 Hz. The predicted peak accelerations ranged from 0.32%g to 0.42%g with the Design Guide Method being the smallest.

For System 10, the predicted fundamental frequencies of the system ranged from 3.58 Hz to 4.41 Hz which are 19% of each other. The predicted peak accelerations ranged from 0.34%g to 0.44%g with the Design Guide Method being the smallest.

Table 4.5 – Summary of Two-way Asymmetric System Results

System	Analysis Method	Effective Weight, lb	Frequency		Acceleration		Remarks
			f_n, Hz	% Difference	a_p/g (%)	% Difference	
System 8	Design Guide	90967	4.54	-	0.33	-	B_j = 34.77 ft, B_g = 26.67 ft
	Design Guide Alt.	76363	4.54	0.0%	0.40	-19.1%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	85004	4.53	0.3%	0.36	-7.0%	
	Analytical	85457	4.52	0.6%	0.35	-6.4%	
	SAP2000	-	4.97	-9.4%	0.41	-23.3%	
System 9	Design Guide	94776	4.54	-	0.32	-	B_j = 34.77 ft, B_g = 26.67 ft
	Design Guide Alt.	76363	4.54	0.0%	0.40	-24.1%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	82838	3.99	12.3%	0.37	-14.4%	
	Analytical	74007	4.22	7.2%	0.41	-28.1%	
	SAP2000	-	4.80	-5.7%	0.42	-32.3%	
System 10	Design Guide	89296	4.41	-	0.34	-	B_j = 34.77 ft, B_g = 26.67 ft
	Design Guide Alt.	74645	4.41	0.0%	0.41	-19.6%	B_j = 30 ft, B_g = 20 ft
	Rayleigh	81591	4.11	6.8%	0.37	-9.4%	
	Analytical	68784	3.58	18.8%	0.44	-29.8%	
	SAP2000	-	4.32	2.1%	0.42	-24.1%	

Note: Effective weight is $2m_eg$.

4.7 SUMMARY

In this chapter, five analysis methods were used to analyze three types of systems to try to find sources of differences present between the Design Guide method and the structural analysis program method. For one-way systems, which include only the beam or girder mode, the Design Guide Method (with modifications for effective mass), the structural analysis program method, the Rayleigh Method, and the Analytical Method all predict approximately the same fundamental frequencies and peak accelerations. All of the methods except the Design Guide method do not account for dissipation in the floor system; therefore, to get correlation of the results, that adjustment was made in the Alternate Design Guide Method.

When two-way symmetric systems are considered, all of the methods predict the fundamental frequency relatively close to each other; however, the accelerations are scattered. When two-way asymmetric systems are considered, both the accelerations and predicted frequencies are scattered. Much of the discrepancies probably result from the differences in the mode shape. The Design Guide Method, Rayleigh Method, and the Analytical Method assume that the mode shape is approximately the deflected shape of the floor under gravity loading. On the other hand, the structural analysis program method solves an eigenvalue and eigenvector problem to obtain the frequencies and mode shapes of the system. Another source for the discrepancy between the structural analysis program method and the other methods may be that the other methods assume that the girder is uniformly loaded. In real systems and in the structural analysis program model, the loading is transferred to the girders through the beams as concentrated loads.

CHAPTER V
CONCLUSIONS AND RECOMMENDATIONS

5.1 INTRODUCTION

This study examined the use of computer modeling techniques to predict the response of floor systems due to walking. First, five ideal floors were modeled using five model set-ups and seven loading protocols. The fundamental frequencies and predicted peak accelerations from these analyses where then compared to predicted fundamental frequency and peak accelerations obtained using the Design Guide procedure. From the results, the best combination of model set-up and loading protocol that produces results that best match the Design Guide procedure results was determined. This combination was then used to model 16 actual floor systems. The fundamental frequencies from the analyses were compared to those experimentally measured and those predicted by the Design Guide procedure. The predicted peak accelerations from the structural analysis program were compared to the predicted peak accelerations from the Design Guide procedure as well as subjective analyses.

The study then explored possible discrepancies between the Design Guide Method and four analysis methods including the Design Guide Alternate Method, the Rayleigh Method, the Analytical Method, and the structural analysis program method. Four one-way systems, three symmetric two-way systems, and three asymmetric two-way systems were analyzed using the four methods. Results include the fundamental frequency and peak acceleration for each system.

5.2 CONCLUSIONS

5.2.1 Model Set-up and Loading Protocol

In general, the best combination of a model set-up and loading protocol for use in a structural analysis program is the Full Bay Strip Model and the Design Guide Loading Function. The Full Bay Strip Model, on average, predicts a fundamental frequency that is 10% greater than that from the Design Guide procedure. When used in conjunction with the Design Guide Function loading, on average the predicted peak acceleration is 78% of the peak acceleration predicted by the Design Guide procedure.

5.2.2 Case Study Buildings

Frequency. In general, the structural analysis program predicts a fundamental frequency that better matches the measured fundamental frequency than does the Design Guide procedure. The predicted fundamental frequency from the Design Guide procedure is 82.6% of the measured frequency on average. However, the predicted fundamental frequency from the structural analysis program is closer to the measured frequency, on average 92.8% of the measured frequency.

Acceleration. In general, the accelerations from the structural analysis program procedure do not correlate well to those from the Design Guide procedure. The Design Guide procedure tends to correlate more closely with the subjective evaluations. On the other hand, the peak accelerations from the structural analysis program procedure do not correlate well with the subjective evaluations. Therefore, the structural analysis program procedure is not an effective procedure to match the Design Guide procedure's peak accelerations, or to predict the acceptability of a floor system when compared to subjective analyses.

5.2.3 Analytical Discrepancies

In general, all of the five different analysis methods predict fundamental frequencies that are within 10% of each other for each system. On the other hand, the effective mass for each of the analysis methods varied as much as 50% from each other. This results in predicted peak accelerations that are as much as 50% different from each other. Much of the error can probably be attributed to the mode shapes used in the analyses. Except for the structural analysis program procedure, the other procedures use the deflected shape as the mode shape. The structural analysis program uses eigenvalues and eigenvectors to obtain the mode shape. Also, except for the structural analysis program method, the analytical methods assume that the girders are uniformly loaded. In the structural analysis program, the loads are transferred from the uniform load on the slab to the girders by the beams.

5.3 RECOMMENDATIONS FOR FUTURE RESEARCH

This study has shown that the structural analysis program is an effective tool to predict the fundamental frequency of a floor system. However, in general, the structural analysis program does not predict peak accelerations that match with those from the Design Guide procedure which correlate with subjective analysis. Some possible areas of research that need to be investigated are presented here, as well as the recommendations of this study.

Effective Mass. The effective mass of a floor system needs to be determined experimentally. Once the effective mass is determined, it must be compared to the effective mass from the Design Guide procedure to verify its accuracy. Once the effective mass and effective stiffness of a floor system is known, the results from the structural analysis program can be verified. Since the effective mass is inversely proportional to the predicted peak acceleration, once the effective mass is understood, the discrepancies in the predicted peak accelerations should be understood.

Acceleration Response. The response of a floor system under a known dynamic loading needs to be measured experimentally and then compared to the acceleration response from a structural analysis program. The structural analysis program results should include results from modal and numerical analyses. Discrepancies between the measured and computer modeling must then be resolved.

Material Damping. The use of material damping needs to be investigated. Material damping may be a useful way of modeling dispersion of vibrations throughout the floor system.

Non-Structural Components. The effect of non-structural components on the vibration characteristics of a floor system needs to be explored. The Design Guide procedure as well as the structural analysis program procedure neglects the presence of non-structural components. These components can stiffen the floor as well as add effective damping to the system.

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APPENDIX A

DESIGN GUIDE PROCEDURE EXAMPLE

A detailed example of the Design Guide procedure follows. The floor system used in this example is the same as System 5 in Section 4.5. Figure A.1 shows details of the system. The beams are W18x35 spaced at 5 ft on center and are supported by W24x62 girders. For this example the system is assumed to have a floor length of 40 ft and a width of 120 ft. The floor has a dead load of 4 psf in addition to the self weight of the system and a live load of 11 psf. The damping for the system is 3% critical. None of the members are assumed to be stiffened by exterior walls.

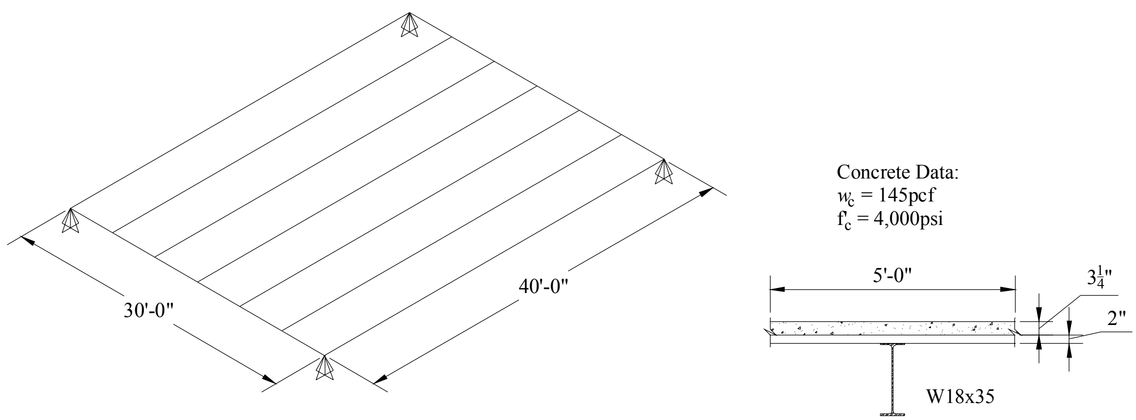

Figure A.1 – Detail of Example Floor

Loading Data:

Slab + 1.0 psf Deck	=	52.4	psf
Dead Loads	=	4.0	psf
Collateral Loads	=	0.0	psf
Live Loads	=	11.0	psf

Actual Beam and Girder Weights

Tributary width for girder = 40.00 / 2 = 20.00 ft

Concrete/Slab Data:

Concrete	d_c = 5.25 in.		f'_c = 4.0 ksi
	wt = 145 pcf		E_c = 3492 ksi

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

Deck Height = 2 in.

Effective concrete thickness in deck = 1 in.

Beam Calculations:

W18x35 $d = 17.70$ in. $A = 10.30\ in^2$

$I_x = 510\ in^4$ $S = 60.0$ in.

$L_b = 40.00$ ft

Uniform Load:

$w_b = (52.4 + 4.0 + 0.0 + 11.0) \times 60.0 / 12 + 35$

$w_b = 372.0$ plf

Transformed Moment of Inertia

Effective Concrete Width	= 60.0	in.
Effective Concrete Depth	= 3.25	in.
Transformed Concrete Width	= 9.75	in.
Transformed Concrete Area	= 31.70	in^2
Distance to Neutral Axis	= 9.42	in.
Transformed Moment of Inertia	= 1747.7	in^4

$$\delta_b = \frac{5 w_b L_b^4}{384 E_s I_b} = \frac{5 \times 372.0 \times 40.0^4 \times 1728}{384 \times E_s \times 1747.7} = 0.423 \text{ in.}$$

$$f_b = 0.18\sqrt{\frac{g}{\delta_b}} = 0.18\sqrt{\frac{386}{0.423}} = 5.44 \text{ Hz}$$

$C_j = 2.0$

Floor Width = 120.0 ft

$$D_s = \frac{12 d_e^3}{12n} = \frac{12 \times 4.25^3}{12 \times 6.15} = 12.48 \ \ in^4/ft$$

$$D_s = \frac{I_b}{S} = \frac{1747.7}{5.0} = 349.54 \ \ in^4/ft$$

$$B_b = \min(C_j (D_s/D_b)^{0.25} L_b, \tfrac{2}{3} \times 120.0) = 2.0 \times (12.48/349.54)^{0.25} \times 40 = 34.77 \text{ ft}$$

$$W_b = 1.0 \times \left(0.372/5.0\right) \times 34.77 \times 40.0 = 103.5 \text{ Kips}$$

Girder Calculations:

W24x62 $d = 23.74$ in. $A = 18.20$ in^2

$I_x = 1550$ in^4 $L_g = 30.00$ ft

Uniform Load:

$w_g = 20.00 \text{ x } (372.0 / 5.0) + 62.0$

$w_b = 1550.0$ plf

Transformed Moment of Inertia

Effective Concrete Width	= 72.0 in. and 36.0 in.
Effective Concrete Depth	= 3.25 in. and 2.00 in.
Transformed Concrete Width	= 11.70 in. and 5.85 in.
Transformed Concrete Area	= 38.04 in^2 and 11.70 in^2
Distance to Neutral Axis	= 10.89 in.
Transformed Moment of Inertia	=4598.3 in^4

$$\delta_g = \frac{5 w_g L_g^{\ 4}}{384 E_s I_g} = \frac{5 \times 1550.0 \times 30.0^4 \times 1728}{384 \times E_s \times 4598.3} = 0.212 \text{ in.}$$

$$f_g = 0.18\sqrt{\frac{g}{\delta_g}} = 0.18\sqrt{\frac{386}{0.212}} = 7.68 \text{ Hz}$$

$C_j = 1.8$

Floor Width = 40.0 ft

$$D_s = \frac{I_b}{S} = \frac{1747.7}{5.0} = 349.54 \ \text{ in}^4/\text{ft}$$

$$D_g = \frac{I_g}{\text{average } L_b} = \frac{4598.3}{20.0} = 229.91 \ \text{ in}^4/\text{ft}$$

$$B_g = \min(C_j (D_b/D_g)^{0.25} L_g, \tfrac{2}{3} \times 40.0) = 1.8 \times (349.54/229.91)^{0.25} \times 30 = 26.67 \text{ ft}$$

$$W_g = 1.0 \times \left(1.550/_{20.0}\right) \times 26.67 \times 30.0 = 62.0 \text{ Kips}$$

Combined Mode Calculations:

Since the girder span, L_g = 30 ft, is less than B_b = 34.77 ft, the girder deflection is reduced by a factor equal to $L_g/B_b \geq 0.5$.

$\max(L_g / B_b, 0.5) = \max(30.00 / 34.77, 0.5) = 0.863$

$\delta_g = 0.863 \times 0.212 = 0.183$ in.

$$f_n = 0.18\sqrt{\frac{g}{\delta_g + \delta_b}} = 0.18\sqrt{\frac{386}{0.183 + 0.423}} = 4.54 \text{ Hz}$$

$$W = \frac{\delta_b}{\delta_b + \delta_g} W_b + \frac{\delta_g}{\delta_b + \delta_g} W_g = \frac{0.423}{0.606} \times 103.5 + \frac{0.183}{0.606} \times 62.0$$

W = 90967 lb

$\beta = 0.03$

$$\frac{a_p}{g} = \frac{65e^{-0.35 f_n}}{\beta W} = \frac{65e^{-0.35 \times 4.54}}{0.03 \times 90967} = 0.49\%$$

System Results

Predicted Fundamental Frequency = 4.54 Hz

Predicted Peak Acceleration = 0.49% g

APPENDIX B

STRUCTURAL ANALYSIS PROGRAM PROCEDURE EXAMPLE

A detailed example of the structural analysis program procedure follows. The floor system used in this example is the same as System 5 in Section 4.5 and Appendix A. Figure B.1 shows details of the system. The beams are W18x35 spaced at 5 ft on center and are supported by W24x62 girders. For this example the system is assumed to have a floor length of 40 ft and a width of 120 ft. The floor has a dead load of 4 psf in addition to the self weight of the system and a live load of 11 psf. The damping for the system is 3% critical. None of the members are assumed to be stiffened by exterior walls.

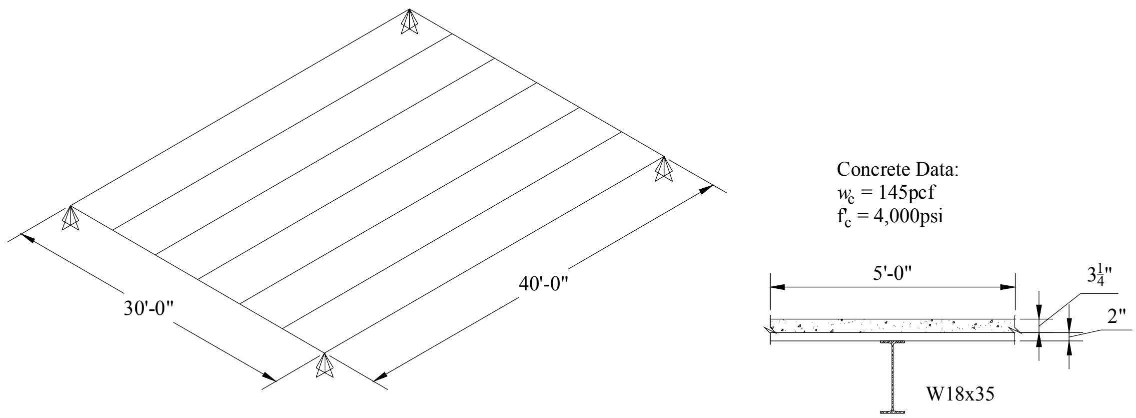

Figure B.1 – Detail of Example Floor

The procedure to model this system in SAP2000 is as follows:

1) **Create New File.** In SAP2000, select the "New Model" option under the file menu. In the options window, choose the "Grid Only" template. Once this is selected, a new window will appear in which the number and spacing of the grid lines must be entered. For this example, the numbers of grid spaces are 1, 1, and 0 for the X, Y, and Z directions, respectively. The grid spacings are 30 ft, 40 ft, and 40 ft for the X, Y, and Z directions, respectively. In SAP2000, since inches are the default length, 30 ft can be entered as "360" or "30ft" and SAP2000 will convert it to inches.
2) **Define Materials.** Define the unit weight, unit mass, modulus of elasticity, and Poisson's ratio for the "VIBCON" material. The unit weight and unit mass are

determined using Equations 2.1 and 2.2. For this example the unit weight is $1.439\text{x}10^{-4}$ kip/in^3, the unit mass is $3.728\text{x}10^{-7}$ kip-s^2/in^4, the modulus of elasticity is 4714.3 ksi, and the Poisson's ratio is 0.2.

3) **Define Frame Sections.** Define frame sections to correspond to each of the framing members, e.g. the beam and girder. The area of the section is equal to the area of the bare steel and the material assigned to the section is "STEEL." The torsional constant and the moment of inertia about the "2" axis are assigned a value of 1.0. Both shear areas are all assigned a value of 0.0. The moment of inertia about the "3" axis is assigned the values of the transformed moment of inertia less the moment of inertia of the slab about its own centroid. For this example, the areas are 10.30 in^2 and 18.20 in^2 for the beam and girder sections, respectively. The moment of inertias about the "3" axis for the beam and girder are:

Beam: $I_{SAP} = I_b - \frac{1}{12} b_e d^3 = 1747.7 - \frac{1}{12} \times 9.754 \times 3.25^3 = 1719.8 \text{in}^4$

Girder: $I_{SAP} = I_g - \frac{1}{12} b_e d^3 = 4598.3 - \frac{1}{12} \times 11.704 \times 3.25^3 = 4564.8 \text{in}^4$

4) **Define Area Sections.** Define a frame section named "SLAB" that is assigned the material VIBCON and a material angle of 0. The area type is set to "Shell" with the membrane and bending thickness set to the effective concrete depth. In this case, the effective concrete depth is 3.25 in.

5) **Draw Frame Elements.** Draw frame elements that are assigned the frame section corresponding to the girder along the X direction grid lines. Then draw frame elements that are assigned the frame section that corresponds to the beams along the Y direction grid lines. All of the sections should have "pinned" moment releases. After the perimeter frame elements are drawn, use the "Quick Draw Secondary Beams" command to draw the remaining beams. After all frame elements are drawn, assign a "Frame Auto Subdivide" to the elements. For this example, the auto subdivide was set to 18 for the frames.

6) **Frame Property Modifiers.** Assign each frame element in the model that is an interior beam in the floor system, a frame property modifier of 0.5 for the mass, weight, and moment of inertia about the "3" axis.

7) **Draw Area Elements.** For each bay in the model, draw an area element that encompasses the entire bay. The area should have the SLAB properties assigned to it. Assign the area an "Area Auto Mesh" that is equal to the frame auto subdivide assigned to the frame elements.
8) **Draw Center Node.** Place a node in the center of the bay.
9) **Restraints.** Assign a translational restraint to each degree of freedom at the column locations.
10) **Define Load Cases.** Define a new load case named "Point." The self-weight multiplier is set to 0.
11) **Define Load.** Apply a 1 lb static load acting downward at the center node. The point load should be part of the Point load case.
12) **Modal Analysis.** Run only the "Modal" analysis case. After the analysis is complete, record the period and frequency of the mode with the largest participating mass ratio for the UZ directions (>0.5). This is the fundamental mode. For this example the fundamental period is 0.216 s, and the fundamental frequency is 4.64 Hz. Unlock so that changes may be made to the model, and modify the "Modal" analysis case so that the maximum number of modes is set to 1 and the frequency shift is set to the fundamental frequency. Calculate the amplitude of the forcing function as $65e^{-0.35fn}$, in this case, 12.816 lb.
13) **Define Time History Function.** Define a sinusoidal time history function named "Walk" that has a period equal to the fundamental period. Adjust the number of steps per cycle so that the time step is less than 0.01 s. Adjust the number of cycles so the time history function has a duration of approximately 10 s. The amplitude of the function is 1.0.
14) **Define Time History Analysis Case.** Define a new analysis case named "Walking." This case is a linear time history analysis case with the following parameters: Load Type = Load, Load Name = Point, Function = Walk, Scale Factor is equal to the amplitude calculate in step 12, Number of Output Time Steps = 10000, Output Time Step Size = 0.01, and Damping is set to the assumed value for all modes.
15) **Analyze.** Run the Walking analysis case.

16) **Response Spectrum.** Record the maximum acceleration for the center node in the Z direction. In this case, 2.33 in/s^2. Divide the peak acceleration by g, 386 in/s^2, to obtain the acceleration in %g. For this case, the peak acceleration due to walking is 0.60% g.

For this example, the fundamental frequency was 4.64 Hz. The peak acceleration due to walking is 0.60% g.

APPENDIX C

ANALYTICAL METHOD EXAMPLE

A detailed example of the Analytical Method follows. The floor system used in this example is the same as System 5 in Section 4.5 and Appendices A and B. Figure C.1 shows details of the system. The beams are W18x35 spaced at 5 ft on center and are supported by W24x62 girders. For this example the system is assumed to have a floor length of 40 ft and a width of 120 ft. The floor has a dead load of 4 psf in addition to the self weight of the system and a live load of 11 psf. The damping for the system is 3% critical. None of the members are assumed to be stiffened by exterior walls.

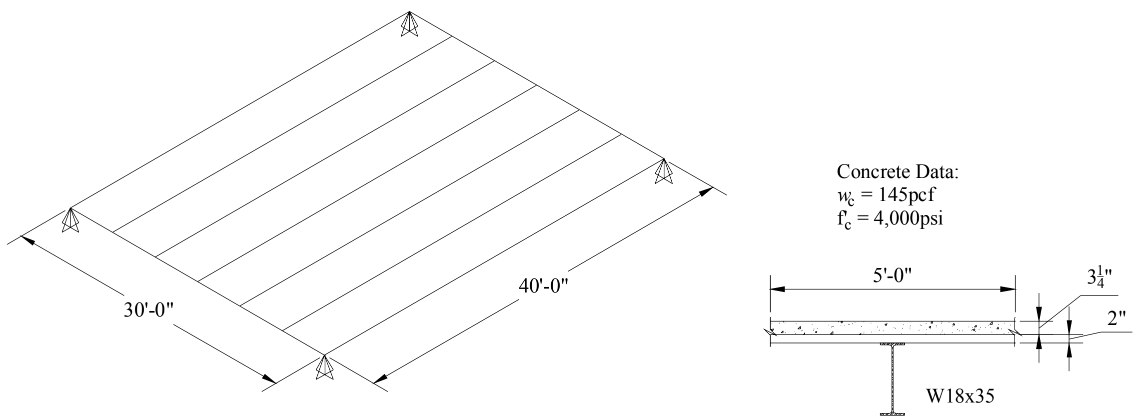

Figure C.1 – Detail of Example Floor

From the Design Guide Procedure:

I_b = 1747.7 in4 $\qquad$ w_b = 372 plf

I_g = 4598.3 in^4 $\qquad$ w_g = 1550 plf

Frequency Calculations:

$\frac{1}{2\pi}\sqrt{\frac{k}{m}}$ can be written as $0.18\sqrt{\frac{g}{\Delta}}$ for a simply supported beam. Therefore, the equation used to calculate the predicted fundamental frequency of the systems is:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_b + \Delta_g}}$$

$$\Delta_b = \frac{5w_bL_b{}^4}{384E_sI_b} = \frac{5\times 372\times 40^4\times 1728}{384\times E_s\times 1747.7} = 0.423 \text{ in.}$$

$$\Delta_g = \frac{5w_gL_g{}^4}{384E_sI_g} = \frac{5\times 1550\times 30^4\times 1728}{384\times E_s\times 4598.3} = 0.212 \text{ in.}$$

Since the beam panel width, L_g, is larger than the girder span, the girder deflection is modified as follows:

$$\Delta_g = \frac{L_g}{L_b}\Delta_g = \frac{30}{40}\times 0.212 = 0.159 \text{ in.}$$

The predicted fundamental frequency then becomes:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_b + \Delta_g}} = 0.18\sqrt{\frac{386}{0.423+0.159}} = 4.64 \text{ Hz}$$

Effective Weight:

Beam Panel Weight:

$$W_b = nNw_bL_b = 1\times 6\times 372\times 40 = 89{,}280 \text{ lb}$$

Here n is the number of bays and N is the number of spaces between beams in a bay.

Girder Panel Weight:

$$W_g = 2w_gL_g\left(1-\frac{S}{L_g}\right) = 2\times 1550\times 30\times\left(1-\frac{5}{30}\right) = 77{,}510 \text{ lb}$$

Here S is the beam spacing in feet.

Combined Panel Weight:

$$\delta_b = \frac{L_b{}^3}{nN48E_sI_b} = \frac{(40\times12)^3}{1\times6\times48\times E_s\times1747.7} = 7.579\times10^{-6}\ \mathrm{lb/in}$$

$$\delta_g = \frac{L_g{}^3}{2\times48E_sI_g} = \frac{(30\times12)^3}{2\times48\times E_s\times4598.3} = 3.645\times10^{-6}\ \mathrm{lb/in}$$

$$\delta = \delta_b + \delta_g = 7.579\times10^{-6} + 3.645\times10^{-6} = 1.122\times10^{-5}\ \mathrm{lb/in}$$

$$W = \frac{\delta_b}{\delta}W_b + \frac{\delta_g}{\delta}W_g = \frac{7.579\times10^{-6}}{1.122\times10^{-5}}\times89280 + \frac{3.645\times10^{-6}}{1.122\times10^{-5}}\times77510 = 85{,}457\ \ \mathrm{lb}$$

Predicted Peak Acceleration:

$$\frac{a_p}{g} = \frac{65e^{-0.35f_n}}{\beta W} = \frac{65e^{-0.35\times4.637}}{0.03\times85457} = 0.50\%$$

For this example, the fundamental frequency is 4.64 Hz. The peak acceleration due to walking is 0.50% g.

APPENDIX D

RAYLEIGH METHOD EXAMPLE

A detailed example of the Rayleigh Method follows. The floor system used in this example is the same as System 5 in Section 4.5 and Appendices A, B, and C. Figure D.1 shows details of the system. The beams are W18x35 spaced at 5 ft on center and are supported by W24x62 girders. For this example the system is assumed to have a floor length of 40 ft and a width of 120 ft. The floor has a dead load of 4 psf in addition to the self weight of the system and a live load of 11 psf. The damping for the system is 3% critical. None of the members are assumed to be stiffened by exterior walls.

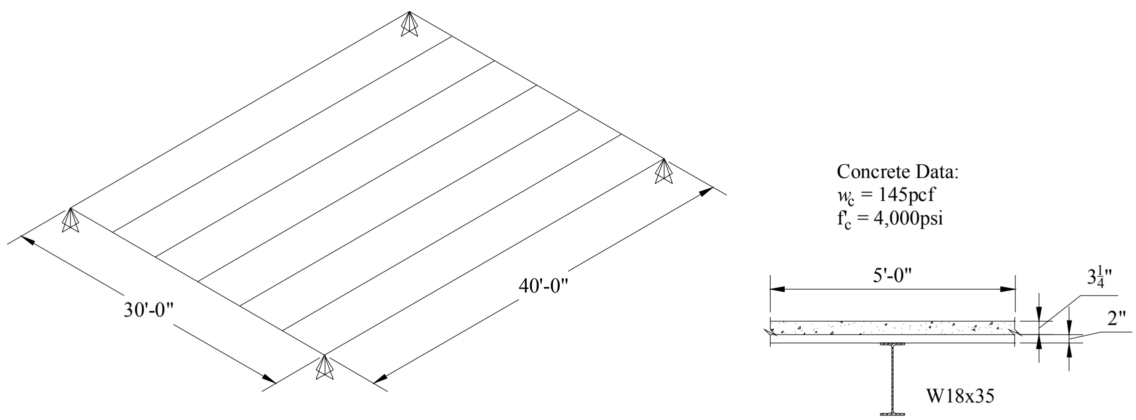

Figure D.1 – Detail of Example Floor

From the Design Guide Procedure:

I_b = 1747.7 in4 $\quad$ w_b = 372 plf

I_g = 4598.3 in^4 $\quad$ w_g = 1550 plf

Frequency Calculations:

The frequency calculations are the same as for the Analytical Method and are as follows:

$$\Delta_b = \frac{5w_b L_b{}^4}{384E_s I_b} = \frac{5\times372\times40^4\times1728}{384\times E_s\times1747.7} = 0.423 \text{ in.}$$

$$\Delta_g = \frac{5w_g L_g{}^4}{384E_s I_g} = \frac{5\times1550\times30^4\times1728}{384\times E_s\times4598.3} = 0.212 \text{ in.}$$

Since the beam panel width, L_g, is larger than the girder span, the girder deflection is modified as follows:

$$\Delta_g = \frac{L_g}{L_b}\Delta_g = \frac{30}{40}\times 0.212 = 0.159\text{ in.}$$

The predicted fundamental frequency then becomes:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_b + \Delta_g}} = 0.18\sqrt{\frac{386}{0.423 + 0.159}} = 4.64\text{ Hz}$$

Effective Weight:

The effective weight of the system is:

$$W = 2\int_0^{L_b}\int_0^{L_g} mg(\phi(x,y))^2\,dxdy$$

where m is the mass per unit area, g is the acceleration due to gravity, and $\phi(x,y)$ is the assumed mode shape.

For this example,

$$mg = \frac{w_b}{S} = \frac{372}{5} = 74.4\text{ psf}$$

$$\phi(x,y) = \frac{\Delta_b}{\Delta_b + \Delta_g}\sin\left(\frac{\pi\cdot y}{L_b}\right) + \frac{\Delta_g}{\Delta_b + \Delta_g}\sin\left(\frac{\pi\cdot x}{L_g}\right)$$

W = 81,758 lb

Predicted Peak Acceleration:

$$\frac{\mathrm{a_p}}{\mathrm{g}} = \frac{65e^{-0.35 f_n}}{\beta \mathrm{W}} = \frac{65e^{-0.35\times 4.637}}{0.03\times 81758} = 0.52\%$$

For this example, the fundamental frequency was 4.64 Hz. The peak acceleration due to walking is 0.52% g.

VITA

(December, 2003)

Jason Daniel Perry was born May 3, 1980 in Hinton, West Virginia to Daniel and Sandra Perry. He grew up near Ronceverte, West Virginia and graduated from Greenbrier East High School in May of 1998. That fall, he entered the Civil Engineering program at West Virginia University Institute of Technology in Montgomery, West Virginia. In May of 2002, he graduated Summa Cum Laude with a Bachelor of Science in Civil Engineering. That fall, he entered the graduate program in the Structural Engineering and Materials Division of the Via Department of Civil and Environmental Engineering at Virginia Polytechnic Institute and State University in Blacksburg, Virginia where he received the Via Fellowship.

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Jason D. Perry