Chapter 4

Comparisons of Measurements to Analytical Predictions

The previous two chapters contained descriptions of the experimental methods and analytical prediction methods used in this research. This chapter describes the floors in detail and presents comparisons of the experimental results and analytical predictions. A summary of the comparisons is at the end of the chapter.

4.1 Long Span Composite Slab Laboratory Specimen

An innovative long-span composite slab system was constructed and vibration tested at the Virginia Tech Structures and Materials Laboratory in 2006. The goal of the original research was to determine if the composite slab system can be used without annoying vibrations to spans of approximately 30 ft and to establish vibration serviceability evaluation procedures for that system.

The specimen is a 30 ft x 30 ft clear span shored composite slab system with beams and girders only at the perimeter. The slab overall thickness is 8-3/4 in. and the deck is 4-5/8 in. deep. Images of the specimen are shown in Figure 4.1. The framing plan, including temporary shoring, is shown in Figure 4.2. The steel deck was spot welded to the W-shape girders and the A992 W-shape girders and beams were connected using snug-tightened bolted shear connections and were supported on stub columns. The stub columns were placed on steel base plates on non-shrink grout to reduce or eliminate base movements. See Figure 4.3 for pictures of these construction details.

Figure 4.1: Long Span Composite Slab Specimen Pictures

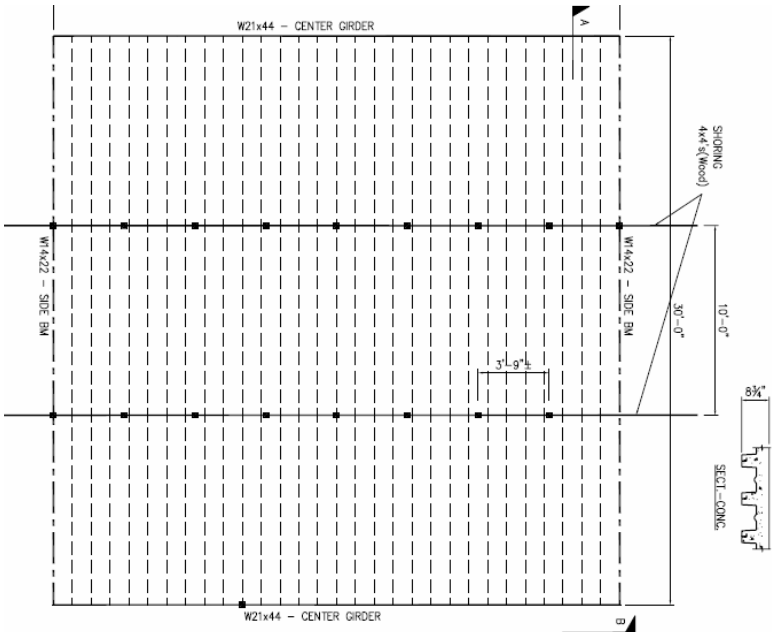


Figure 4.2: Long Span Composite Slab Specimen Framing Plan

Figure 4.3: Long Span Composite Slab Construction Details. (a) Deck Spot Welds; (b) Beam Connection; (c) Stub Column; (d) Base Plate.

4.1.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker was placed in the middle of the specimen and accelerations were measured at the quarter points, edges, and corners. The measurements and predictions both indicate that the structure had one dominant mode that can be excited using one of the first four harmonics of the walking force. The measured and predicted natural frequencies for this natural mode are 4.98 Hz and 5.19 Hz (Predicted/Measured = 1.04). The shape is shown in Figure 4.4. The measured and predicted natural frequency and mode shape are in good agreement.

The measured mode shape is quasi-real as indicated by the starburst plot shown in Figure 4.5. This indicates that the assumption of proportional damping in the model is valid and that the specimen's damping is approximately evenly distributed over the specimen rather than concentrated at joints.

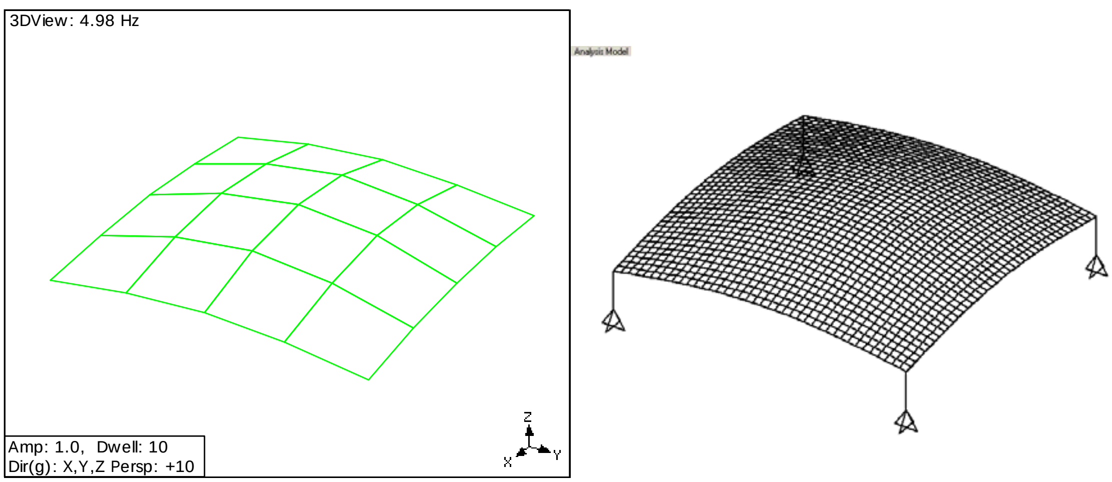


Figure 4.4: Measured and Predicted Mode Shapes

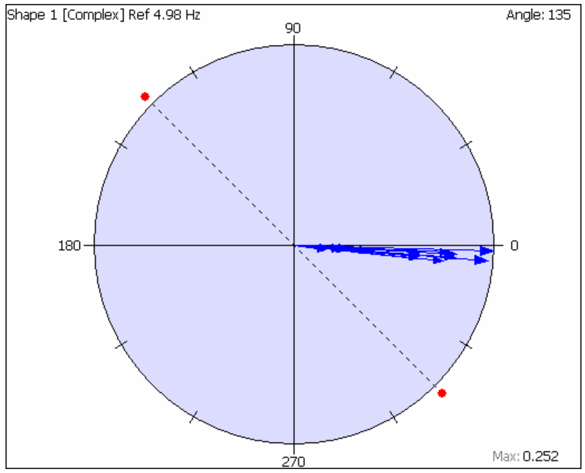


Figure 4.5: Starburst Plot for Long Span Composite Slab Specimen

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping was used in the model to allow a valid comparison. EMA FRF curve-fitting was used to determine the viscous modal damping ratio: 0.436% of critical. The curve-fit FRFs for several DOF are shown in Figure 4.6, indicating a very precise match of the parametric model and estimated FRFs. Constant hysteretic damping with stiffness proportional coefficient equal to 0.00872 (mass proportional coefficient equals zero) was used in the steady-state analysis. The measured and predicted accelerance FRF magnitudes are shown in Figure 4.7. The measured peak magnitude was 0.364 %g/lbf whereas the predicted was 0.437 %g/lbf (Predicted/Measured = 1.20). Over-prediction of the peak magnitude indicates that the model slightly under-predicted the effective mass for this mode, most likely indicating that the specimen contained more concrete than the idealized volume. Slight differences in the measured and predicted mode shapes probably also have contributed to the difference in measured and predicted peak magnitudes.

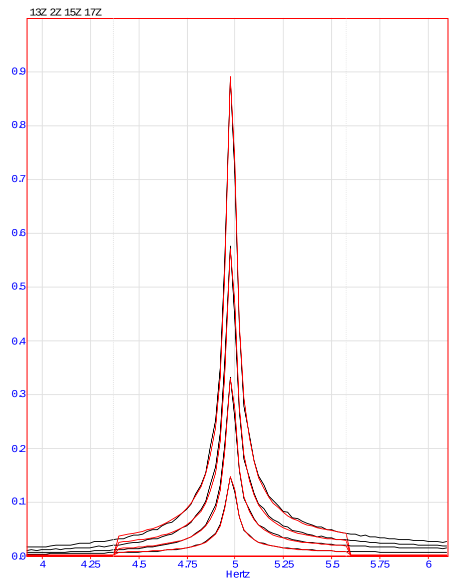


Figure 4.6: Curve-Fit FRFs for Long Span Composite Slab Specimen

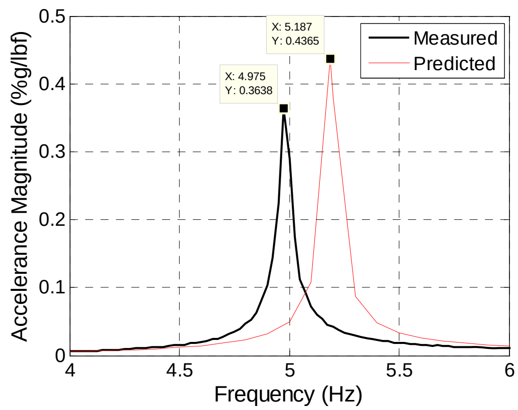


Figure 4.7: Measured and Predicted Accelerance Magnitudes, Long Span Composite Slab Specimen

4.1.2 Response to Walking (Predictions Using Individual Footsteps)

Response to walking parallel and perpendicular to the deck were measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. The step frequency was 1.66 Hz (100 bpm) and 1.73 Hz (104 bpm) during tests and response history analyses, respectively, to cause resonance with the third harmonic of the walking force both in reality and in the model. Measured viscous modal damping was used in the model.

Walking Parallel to Deck. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.8. The measured acceleration spectrum indicates that the third harmonic of the walking force caused resonance. The measured acceleration waveform also indicates a third harmonic resonant build-up. Walking tests were performed at numerous step frequencies ranging from approximately 1.5 Hz (90 bpm) to 1.8 Hz (110 bpm), but these resulted in lower responses. The measured and predicted peak accelerations are 1.36%g and 1.68%g, respectively (Predicted / Measured = 1.24). Presumably, the response is over-predicted because the model under-predicted the effective mass (Predicted / Measured = 1.20) and because walking was very slightly off-resonance.

Walking Perpendicular to Deck. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.9. The measured acceleration spectrum and measured acceleration waveform indicate a third harmonic resonant build-up, although with a lower peak acceleration than was measured for walking parallel to the slab. The measured and predicted peak accelerations are 0.972%g and 1.68%g, respectively (Predicted / Measured = 1.73). Presumably, the response is over-predicted because the model under-predicted the effective mass (Predicted / Measured = 1.20) and because walking was slightly off-resonance. It is not known why the measured response was significantly lower than for walking parallel to the deck, although it is noted that the third harmonic frequency was slightly less than the natural frequency for walking perpendicular whereas it was slightly higher than the natural frequency for walking parallel.

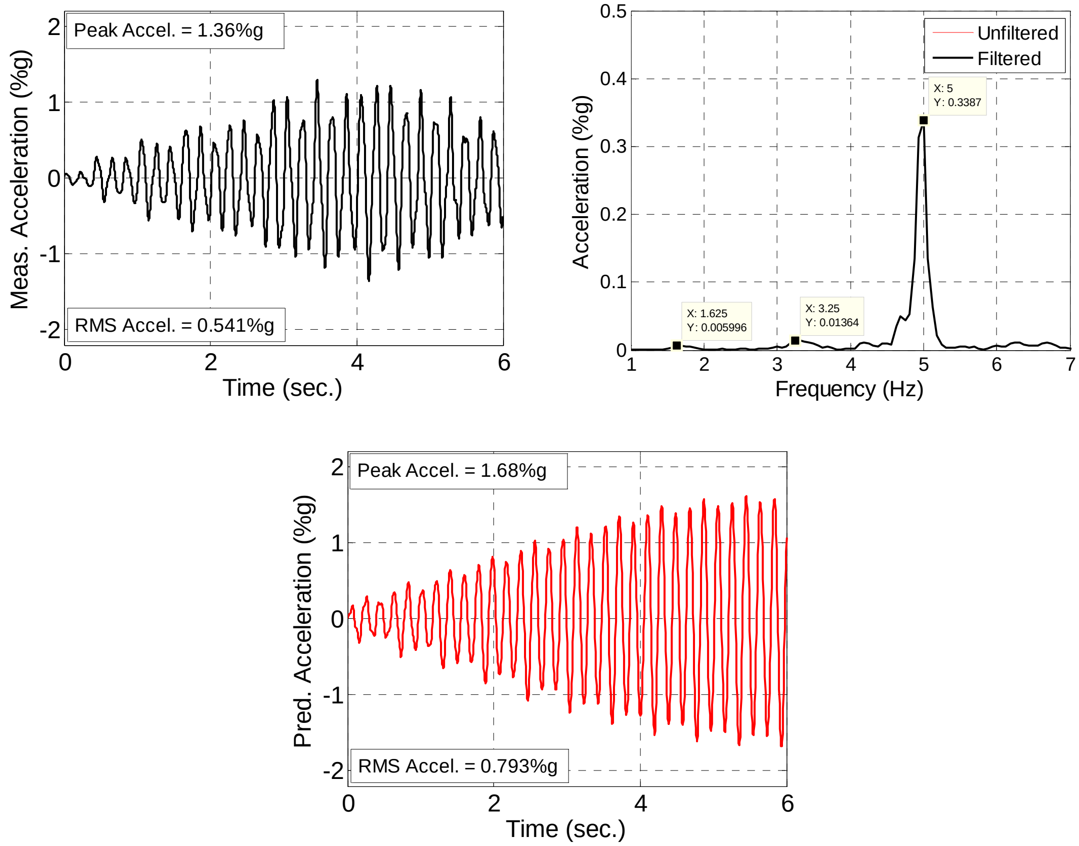


Figure 4.8: Acceleration Response to Walking Parallel to Deck, Individual Footsteps, Long Span Composite Slab Specimen. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

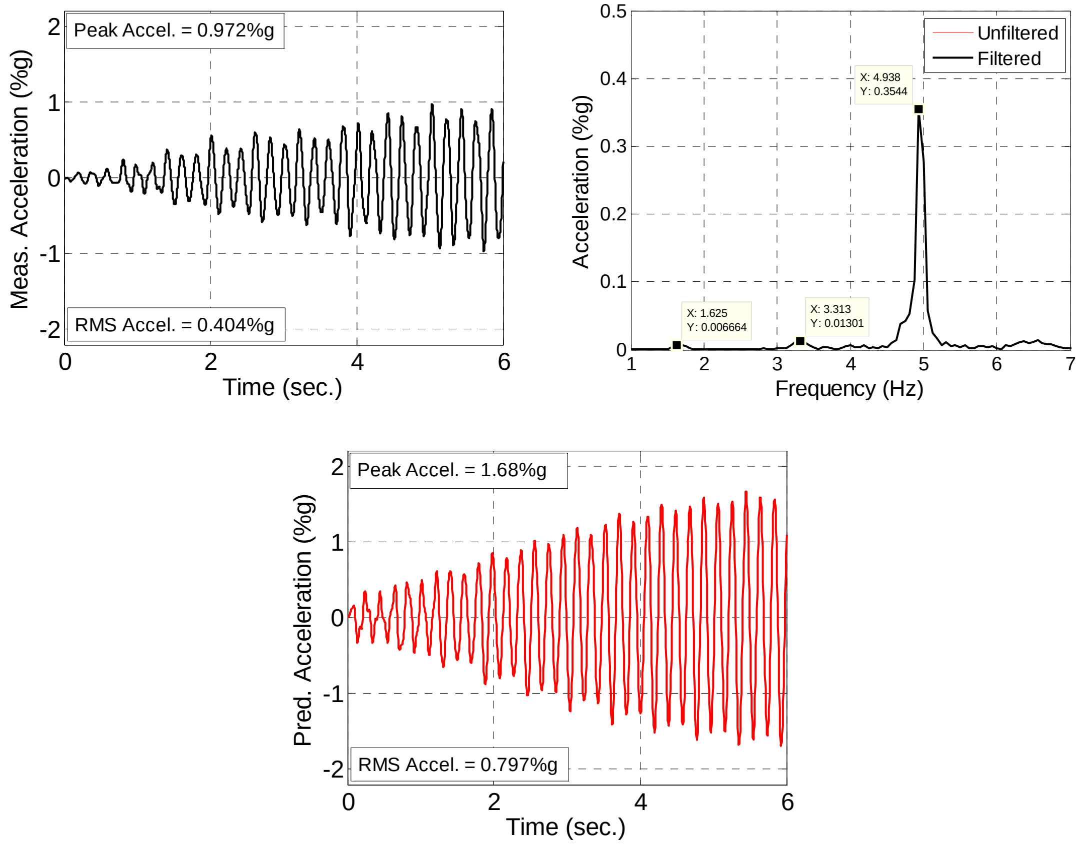


Figure 4.9: Acceleration Response to Walking Perpendicular to Deck, Individual Footsteps, Long Span Composite Slab Specimen. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

4.1.3 Response to Walking (Predictions Using Fourier Series Loading)

Response to walking parallel and perpendicular to the deck were also predicted using the Fourier series load as described in Chapter 3. As was the case for the prediction using individual footsteps, the step frequency was defined so that the third harmonic caused resonance during response history analysis. Analyses were performed using all four Fourier series terms and only using the harmonic that matched the natural frequency. The following paragraphs present the results that are summarized in Table 4.1.

Table 4.1: Measured and Predicted Peak Accelerations, Long Span Composite Slab Specimen, Fourier Series Loading

Description	Peak Acceleration Measured (%g)	Peak Acceleration Predicted (%g)	Peak Acceleration Predicted / Measured
Walking Parallel to Deck, 4 Terms	1.36	2.71	1.99
Walking Parallel to Deck, 1 Term	1.36	2.63	1.93
Walking Parallel to Deck, 4 Terms	0.972	2.71	2.79
Walking Parallel to Deck, 1 Term	0.972	2.63	2.71

Walking Parallel to Deck. The measured walking acceleration waveform for the test with maximum response and the predicted waveform are shown in Figure 4.10. The measured and predicted peak accelerations are 1.36%g and 2.71%g, respectively (Predicted / Measured = 1.99). The response is over-predicted because the entire response history was subjected to loading at mid-bay (the mode shape's anti-node), the model under-predicted the effective mass and because walking was very slightly off-resonance.

Figure 4.11 shows the response history prediction if only the third harmonic sinusoidal load is applied. The predicted peak acceleration, only including this term is 2.63%g, compared with 2.71%g predicted using all four terms (2.71/2.63 = 1.03). The other three harmonics provide a negligible portion of the response, especially considering the coarseness of prediction of floor vibration response using any current method. The predicted-to-measured peak acceleration ratio is 1.93.

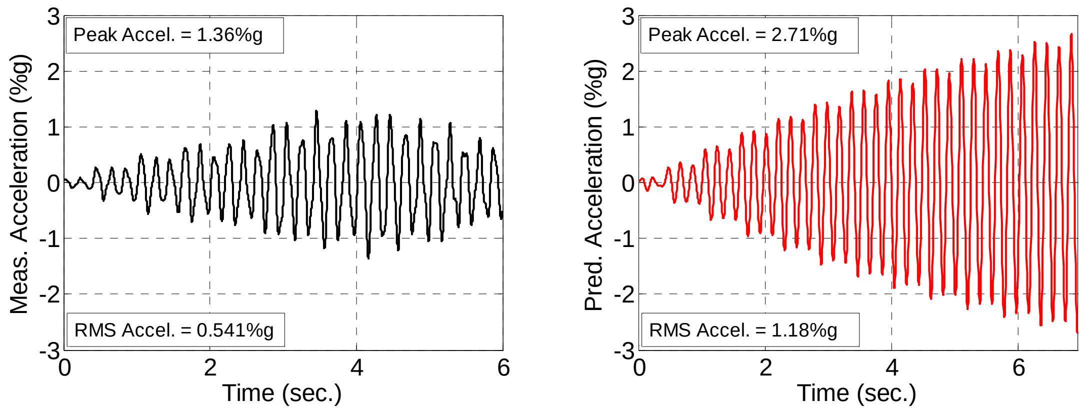


Figure 4.10: Acceleration Response to Walking Parallel to Deck, Fourier Series Loading, Long Span Composite Slab Specimen. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

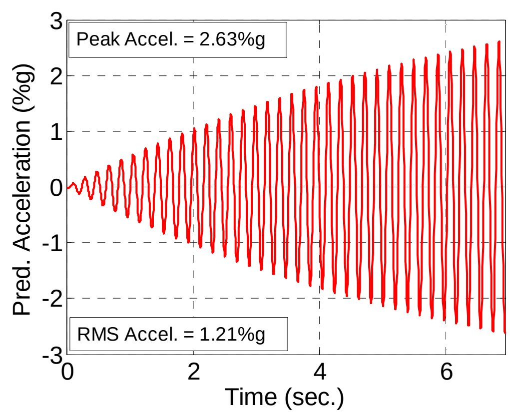
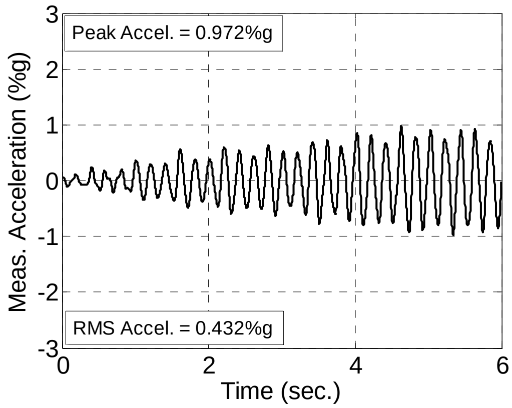


Figure 4.11: Acceleration Response to Walking, Fourier Series, Long Span Composite Slab Specimen (Third Harmonic Force Only)

Walking Perpendicular to Deck. The measured walking acceleration waveform for the test with maximum response and the predicted waveform are shown in Figure 4.12. The measured and predicted peak accelerations are 0.972%g and 2.71%g, respectively (Predicted / Measured = 2.79). The response is over-predicted for the same reasons that are given in the preceding paragraph.



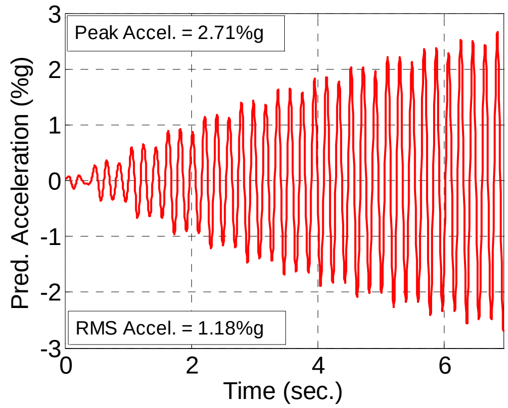


Figure 4.12: Acceleration Response to Walking Perpendicular to Deck, Fourier Series Loading, Long Span Composite Slab Specimen. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

4.1.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. Using this method, the acceleration response to walking is computed using the accelerance FRF magnitude. In general, the predicted FRF magnitude will be available, but the measured one will not. However, for the purposes of this research, both are used to predict the acceleration response to walking.

The predicted accelerance peak magnitude is 0.437 %g/lbf at a frequency of 5.19 Hz. The walking path was 30 ft and the damping ratio was 0.436% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 2.59%g. The measured peak accelerations for walking parallel to the deck and perpendicular to the deck, respectively, are 1.36%g and 0.972%g. The ratio of predicted-to-measured accelerations for the two directions is 1.90 and 2.66, respectively, indicating very significant over-predictions.

The measured accelerance peak magnitude is 0.364 %g/lbf at a frequency of 4.98 Hz. The acceleration is predicted to be 2.11%g. The ratio of predicted-to-measured accelerations for the two directions is 1.55 and 2.17, respectively. Again, the ratios indicate very significant over-prediction.

4.2 Long Span Composite Slab Mockup

Floor vibration tests were performed on a long span composite slab floor mockup in January, 2008 (Figure 4.13) as a continuation of the research described in Section 4.1.

The mockup is an approximately 61 ft by 74 ft. floor, consisting of four 30 ft bays separated by a corridor as would be the case in a hotel. The top of slab is approximately 14 ft above grade. The floor plan is shown in Figure 4.14. The four main bays are constructed using a 9 5/8 in. total thickness composite slab on 4 1/2 in. steel deck. The corridor slab is a 5 1/4 in. total thickness composite slab on 2 in. steel deck. The slab is supported by hot-rolled steel girders only at the column lines. Mechanical and pipe openings were included at the girder lines.

Figure 4.13: Long Span Composite Slab Mockup Picture

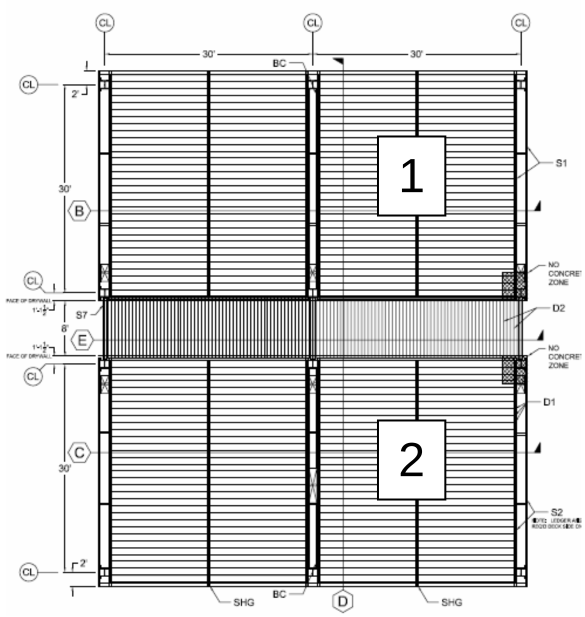


Figure 4.14: Long Span Composite Slab Mockup Framing Plan

4.2.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker was placed in the middle of Bay 1 and 2, indicated in Figure 4.14. An armature accelerometer was used to indirectly measure the applied force. The effective mass to armature mass ratio was adequate to allow only small force glitches, so the errors associated with them are small. Accelerations were measured at the bay centers and midspan of girders and beams.

One very dominant mode was measured when the shaker was in Bay 1 and another was measured when the shaker was in Bay 2. Due to the symmetric geometry and corridor slab, the mockup behaved almost like two separate systems. Table 4.2 lists the measured and predicted natural frequencies. The mode shapes are shown in Figure 4.15. The model very accurately predicted the frequencies and shapes for these four modes. These four mode shapes are quasi-real as shown in Figure 4.16. During tests with the shaker at Location 1, a mode was also detected at 5.89 Hz, but it was not predicted by the model. This mode, shown in Figure 4.17, very closely resembles the 5.75 Hz mode, except with more complexity. It is also more slightly symmetric than the 5.75 Hz mode.

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping was used in the model to allow a valid comparison. EMA FRF curve-fitting was used to determine the viscous modal damping ratios shown in Table 4.3. The curve-fit FRFs for several DOF are shown in Figure 4.6, indicating a very precise match of the parametric model and estimated FRFs.

Interpolated hysteretic damping was used in the steady-state analysis. Mass proportional coefficients were zero. For the FRF with shaker at Location 1, the stiffness proportional coefficients were set to 0.010 at 5.56 Hz and 0.0102 at 6.01 Hz. For the FRF with the shaker at Location 2, the stiffness proportional coefficients were set to 0.0110 at 5.71 Hz and 0.0190 at 6.56 Hz. The measured and predicted accelerance FRF magnitudes are shown in Figure 4.19. Table 4.4 lists the accelerance peak magnitudes. The model reasonably predicted the dominant modes, but not the others. The main discrepancy is frequency spacing. The model predicted frequencies that were much farther apart for Bay 1 and slightly farther apart in Bay 2. In Bay 1, the model also did not predict the measured mode at 5.89 Hz.

Table 4.2: Long Span Composite Slab Mockup Natural Frequencies

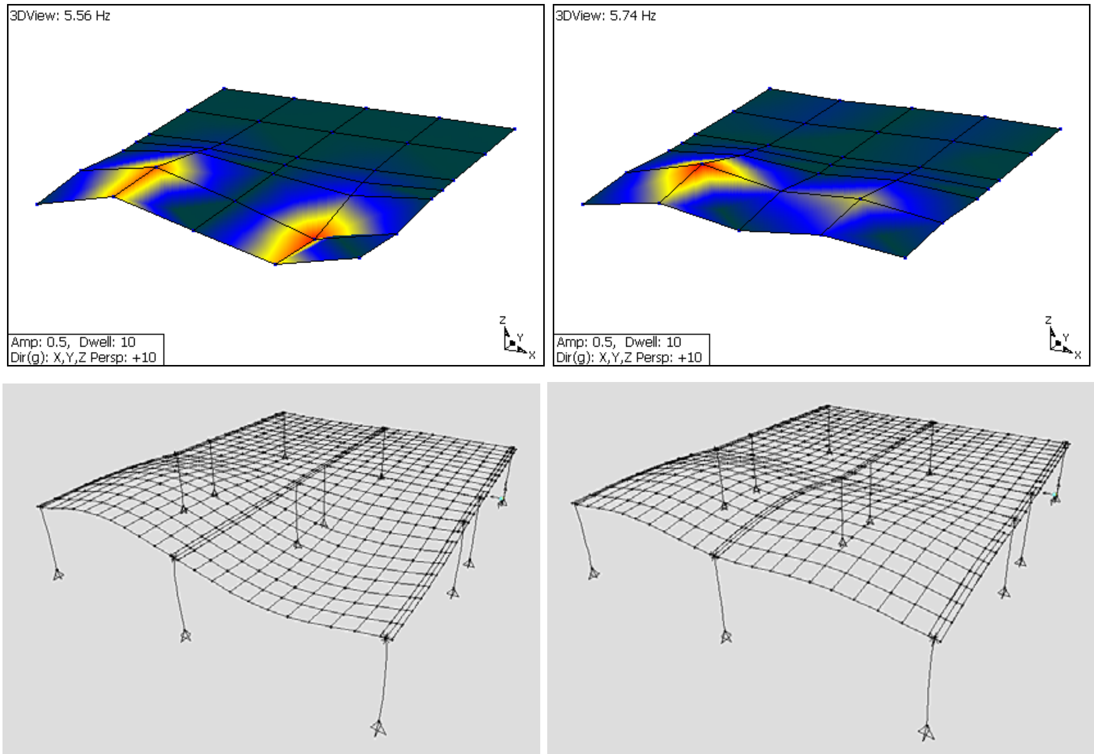
Description	Natural Frequency		
	Measured (Hz)	Predicted (Hz)	Predicted / Measured
Shaker at 1, Mode 1	5.55	5.56	1.00
Shaker at 1, Mode 2	5.75	6.01	1.05
Shaker at 1, Mode 3	5.89	-	-
Shaker at 2, Mode 1	6.00	5.71	0.952
Shaker at 2, Mode 2	6.63	6.56	0.989

Table 4.3: Damping Ratios, Long Span Composite Slab Mockup

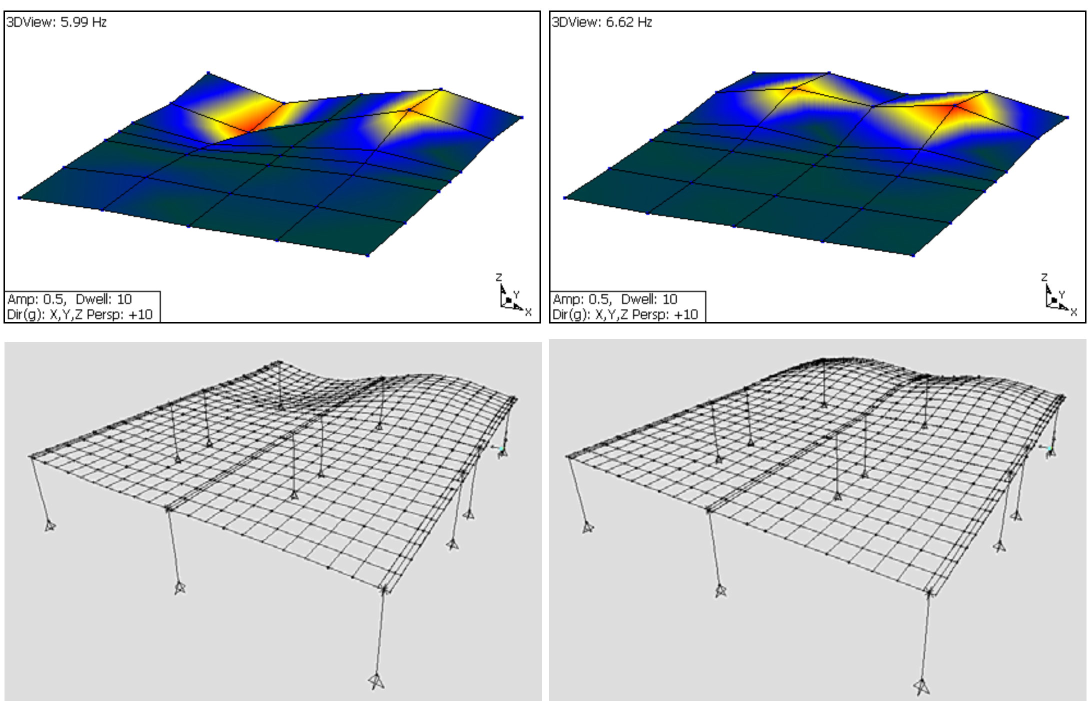
Description	Damping (% of Critical)
Shaker at 1, Mode 1	0.50
Shaker at 1, Mode 2	0.51
Shaker at 1, Mode 3	0.71
Shaker at 2, Mode 1	0.55
Shaker at 2, Mode 2	0.95

Table 4.4: Measured and Predicted Accelerance Peak Magnitudes, Long Span Composite Slab Mockup

Description	Accelerance Peak Magnitude		
	Measured (%g/lbf)	Predicted (%g/lbf)	Pred. / Meas.
Shaker at 1, Mode 1	0.153	0.165	1.08
Shaker at 1, Mode 2	0.0680	0.166	2.44
Shaker at 2, Mode 1	0.162	0.151	0.932
Shaker at 2, Mode 2	0.0638	0.0961	1.51



Meas. = 5.55 Hz (Predicted = 5.56 Hz) **Meas. = 5.75 Hz (Predicted = 6.01 Hz)**



Meas. = 6.00 Hz (Predicted = 5.71 Hz) **Meas. = 6.63 Hz (Predicted = 6.56 Hz)**

Figure 4.15: Mode Shapes, Long Span Composite Slab Mockup

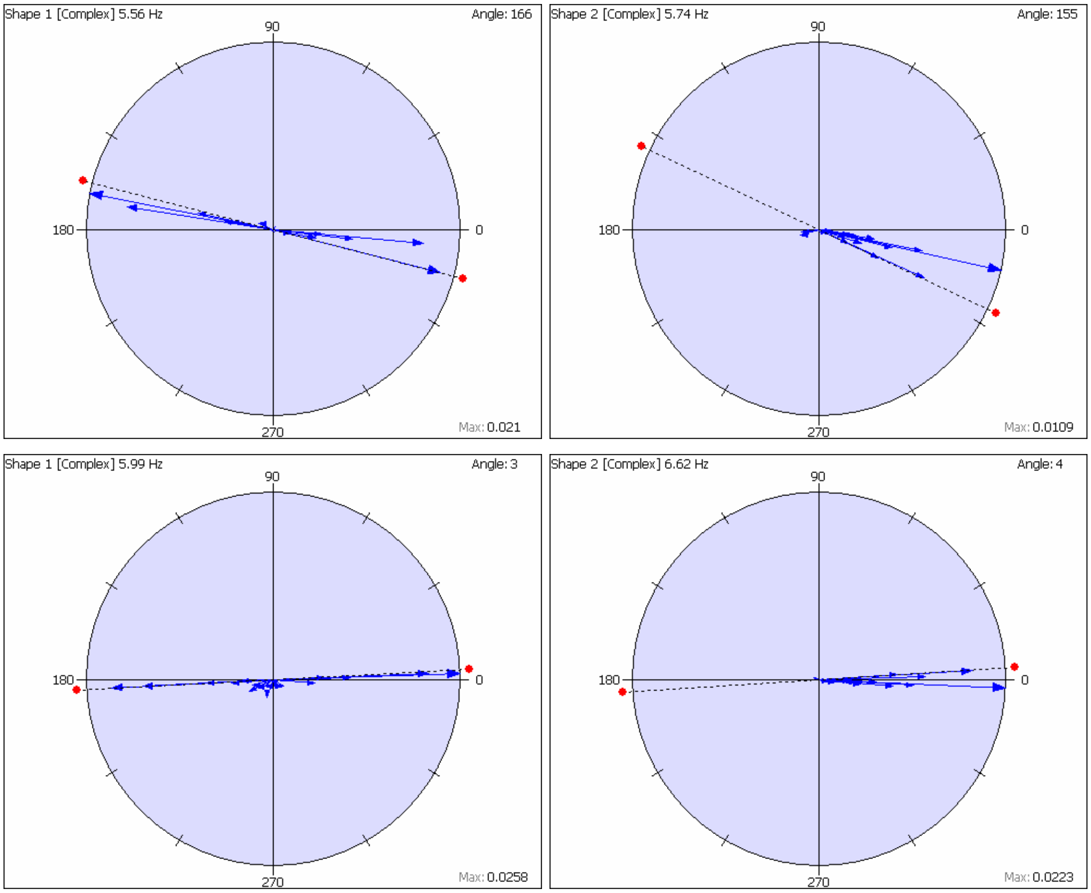


Figure 4.16: Starburst Plots for Long Span Composite Slab Mockup

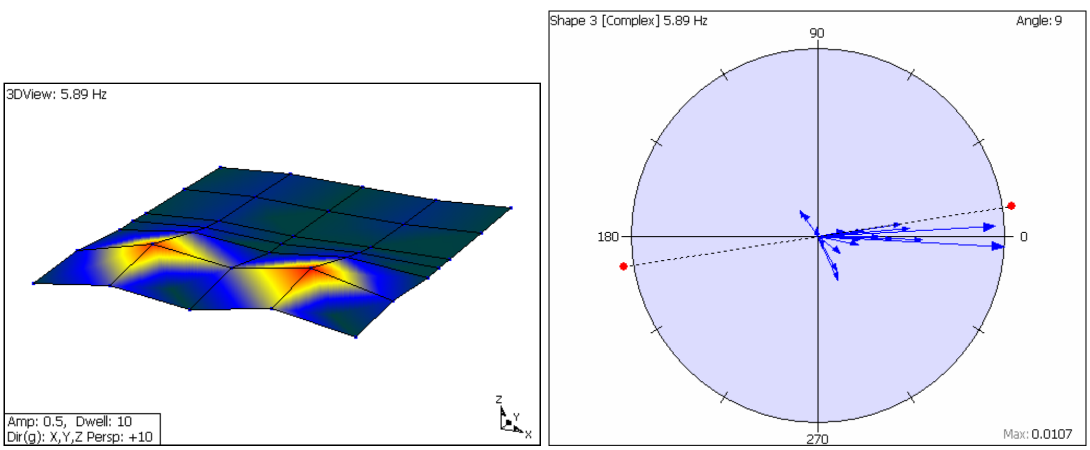


Figure 4.17: Additional Measured Natural Mode, Long Span Composite Slab Mockup

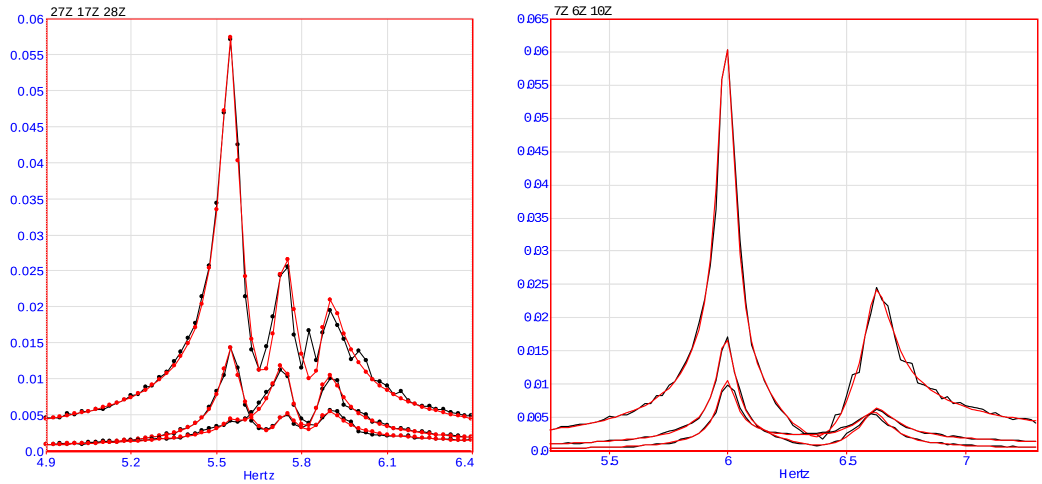


Figure 4.18: Curve-Fit FRFs for Long Span Composite Slab Mockup

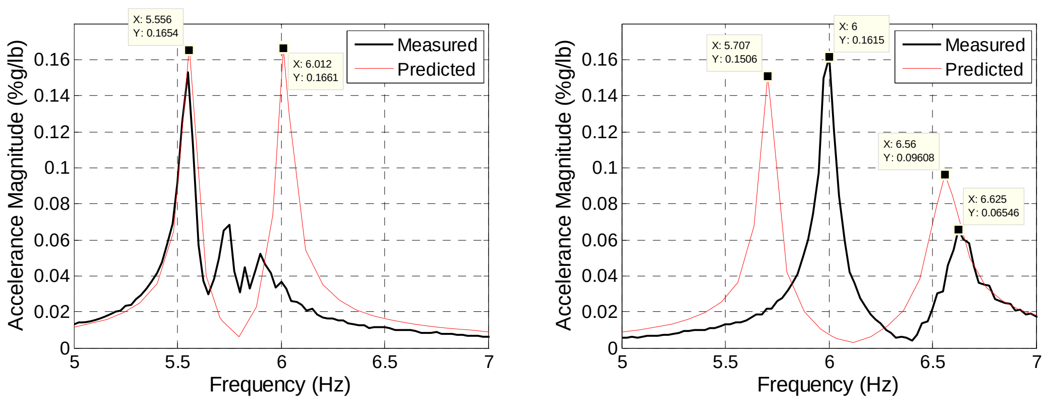


Figure 4.19: Measured and Predicted Acceleration Magnitudes, Long Span Composite Slab Mockup

4.2.2 Response to Walking (Predictions Using Individual Footsteps)

Response to walking parallel and perpendicular to the deck were measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. The step frequency in each bay was the third subharmonic of the dominant frequency, both in reality and in the model. Measured viscous modal damping was used in the model.

Walking in Bay 1. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.20. The measured acceleration spectrum indicates that the third harmonic of the walking force caused resonance. The measured acceleration waveform indicates a very weak third harmonic resonant build-up. The measured and predicted peak accelerations are 0.768%g and 1.18%g, respectively (Predicted / Measured = 1.54).

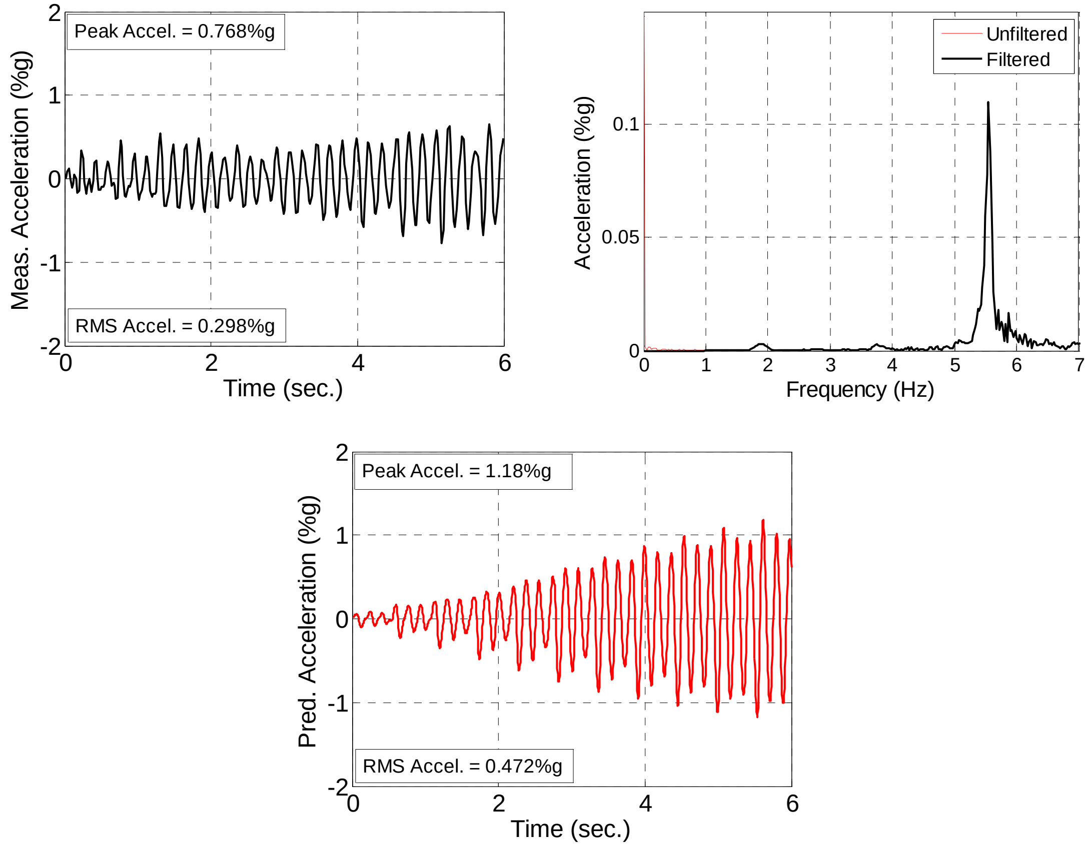


Figure 4.20: Acceleration Response to Walking in Bay 1, Individual Footsteps, Long Span Composite Slab Mockup. (a) Measured Acceleration Waveform (111 bpm); (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

The response is over-predicted because the model predicts that the dominant mode (5.55 Hz) is well separated from other adjacent modes whereas in reality there are two additional modes (5.75 Hz and 5.89 Hz) nearby. Some of the energy caused excitation of these modes, as indicated by the acceleration spectrum. This caused a much less complete resonant build-up during the tests than was predicted by the model. This illustrates the main difficulty in predicting the dynamic response of floors. The finite element model predicted natural frequencies and mode shapes that were in excellent agreement with the measured results (frequencies within 5% and similar shapes for the first 4 out of 5 measured modes). However, it could not accurately enough predict the relative subtleties of frequency spacing and presence of the third mode in Bay 1. These currently unavoidable errors caused the model to significantly over-predict the response.

Walking in Bay 2. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.21. The measured acceleration spectrum indicates that the third harmonic of the walking force caused resonance. The measured acceleration waveform indicates a third harmonic resonant build-up. The measured and predicted peak accelerations are 0.966%g and 0.790%g, respectively (Predicted / Measured = 0.818).

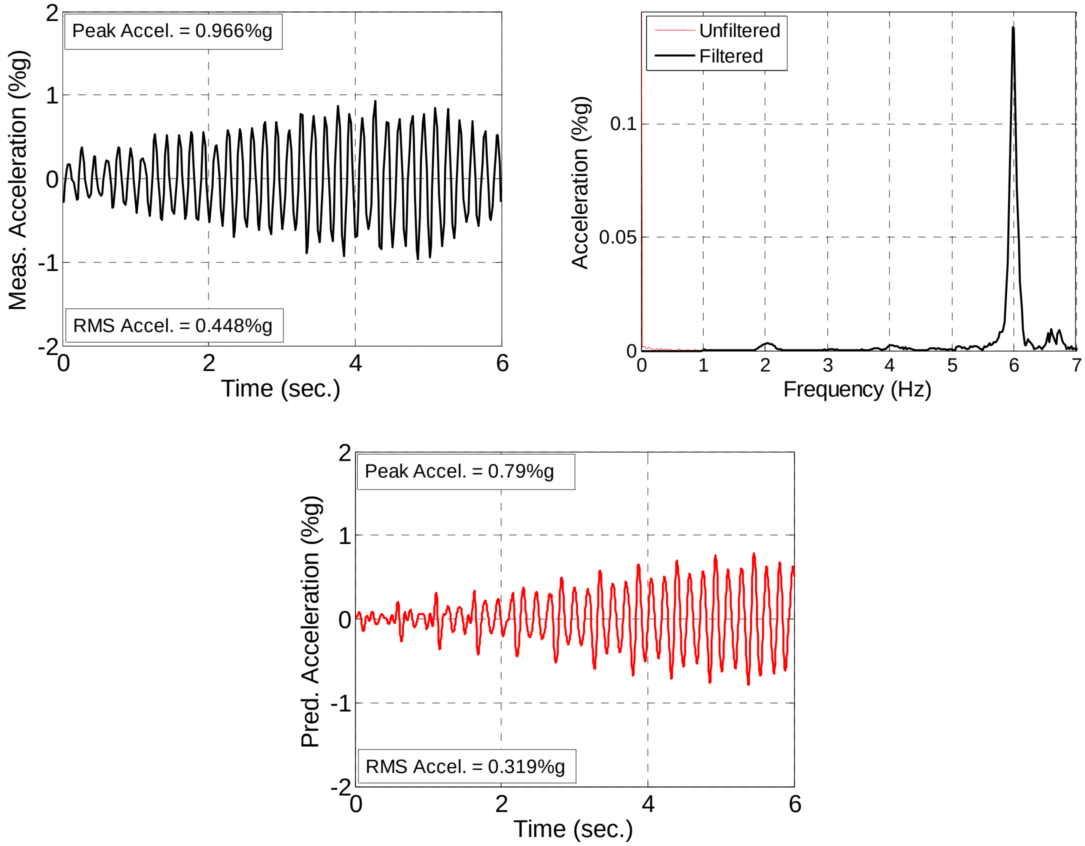


Figure 4.21: Acceleration Response to Walking in Bay 2, Individual Footsteps, Long Span Composite Slab Mockup. (a) Measured Acceleration Waveform (120 bpm); (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

In this bay, the model more accurately predicted the acceleration response because it more accurately predicted the modal properties. The measured modes were spaced apart far enough to allow a resonant build-up, so the actual behavior was easier to predict.

4.2.3 Response to Walking (Predictions Using Fourier Series Loading)

Response to walking in Bay 1 and Bay 2 were also predicted using the Fourier series load as described in Chapter 3. As was the case for the prediction using individual footsteps, the step frequency was defined so that the third harmonic caused resonance during response history analysis. Analyses were performed using all four terms of the Fourier series and using only the harmonic that matches the natural frequency. The following paragraphs present the results which are summarized in Table 4.5.

Table 4.5: Measured and Predicted Peak Accelerations, Long Span Composite Slab Mockup, Fourier Series Loading

	Peak Acceleration		
Description	Measured (%g)	Predicted (%g)	Predicted / Measured
Walking in Bay 1, 4 Terms	0.768	1.38	1.80
Walking in Bay 1, 1 Term	0.768	1.28	1.67
Walking in Bay 2, 4 Terms	0.966	1.39	1.44
Walking in Bay 2, 1 Term	0.966	1.27	1.31

Walking in Bay 1. The measured walking acceleration waveform for the test with maximum response and the predicted waveform are shown in Figure 4.22. The measured and predicted peak accelerations are 0.768%g and 1.38%g, respectively (Predicted / Measured = 1.80).

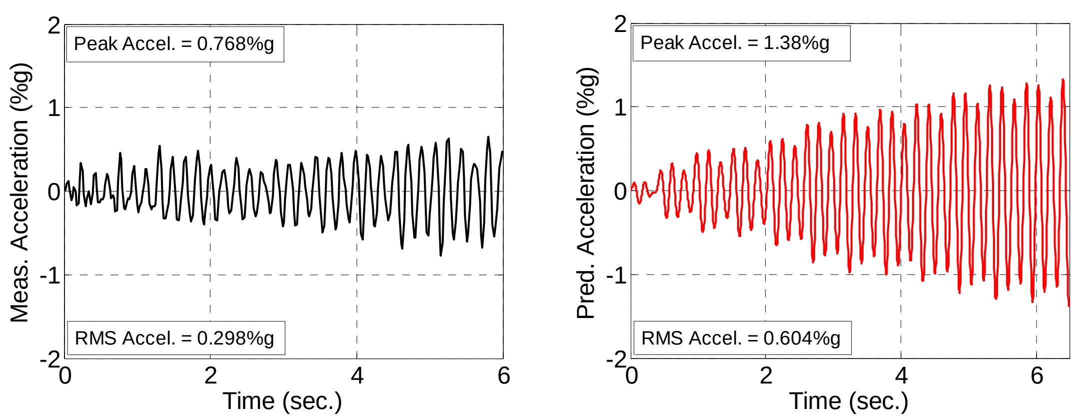


Figure 4.22: Acceleration Response to Walking in Bay 1, Fourier Series Loading, Long Span Composite Slab Mockup. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

Figure 4.23 shows the response history prediction if only the third harmonic sinusoidal load is applied. The predicted peak acceleration, only including this term is 1.28%g, compared with 1.38%g predicted using all four terms (1.38/1.28=1.08). The other three harmonics provide a small portion of the predicted response in this case.

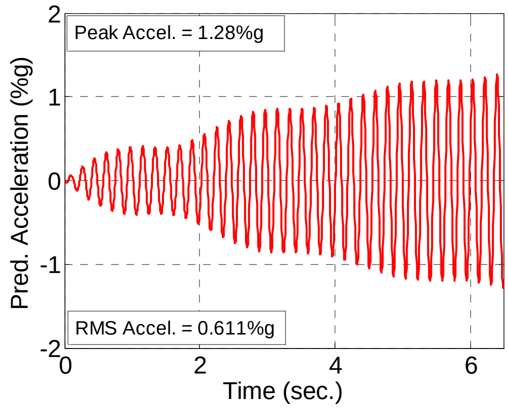


Figure 4.23: Acceleration Response to Walking in Bay 1, Fourier Series (Third Harmonic Force Only), Long Span Composite Slab Mockup

Walking in Bay 2. The measured walking acceleration waveform for the test with maximum response and the predicted waveform are shown in Figure 4.22. The measured and predicted peak accelerations are 0.966%g and 1.39%g, respectively (Predicted / Measured = 1.44).

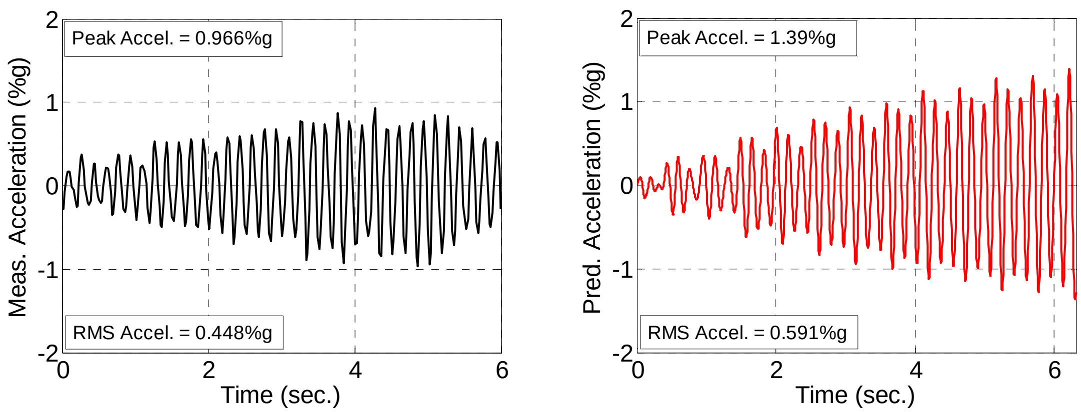


Figure 4.24: Acceleration Response to Walking in Bay 2, Fourier Series, Long Span Composite Slab Mockup

Figure 4.25 shows the response history prediction if only the third harmonic sinusoidal load is applied. The predicted peak acceleration, only including this term is 1.27%g, compared with 1.39%g predicted using all four terms (1.39/1.27=1.09). The other three harmonics provide a small portion of the predicted response in this case.

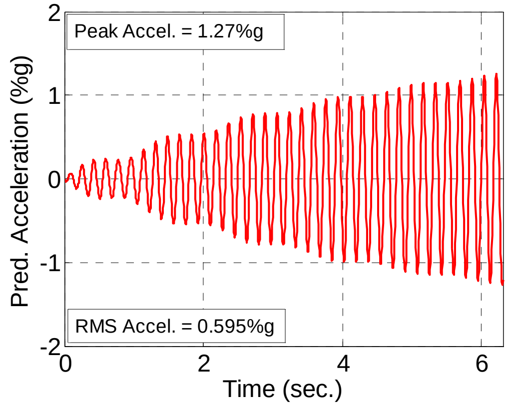


Figure 4.25: Acceleration Response to Walking in Bay 2, Fourier Series (Third Harmonic Force Only), Long Span Composite Slab Mockup

4.2.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. The response was predicted using the predicted FRF magnitude and the measured FRF magnitude.

In Bay 1, the predicted accelerance peak magnitude is 0.162 %g/lbf at a frequency of 5.56 Hz. The walking path was 30 ft and the damping ratio was 0.50% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 1.19%g whereas the measured peak acceleration due to walking was 0.768%g. The ratio of predicted-to-measured accelerations is 1.55.

In Bay 1, the measured accelerance peak magnitude is 0.153 %g/lbf at a frequency of 5.55 Hz. The acceleration is predicted to be 1.12%g. The ratio of predicted-to-measured accelerations is 1.46. The response in Bay 1 was significantly over-predicted using both the predicted and measured FRF magnitude.

In Bay 2, the predicted accelerance peak magnitude is 0.136 %g/lbf at a frequency of 5.72 Hz. The walking path was 30 ft and the damping ratio was 0.55% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 1.06%g whereas the measured peak acceleration due to walking was 0.966%g. The ratio of predicted-to-measured accelerations is 1.10.

In Bay 2, the measured accelerance peak magnitude is 0.162 %g/lbf at a frequency of 6.00 Hz. The acceleration is predicted to be 1.30%g. The ratio of predicted-to-measured accelerations is 1.35. The response in Bay 2 was accurately predicted using the predicted FRF and slightly over-predicted using the measured FRF.

4.3 Square-End Joist Footbridge

A single span square-end joist footbridge was constructed and vibration tested at the Virginia Tech Structures and Materials Laboratory in 2005 and 2006. The goal of the original research was to determine if typical construction details which connect the top and bottom chord to CMU walls decreases vibration response. The footbridge was constructed in three stages, but only the first stage, which is nominally simply supported, is included in this research. Modal test results from the other two stages are not suitable for direct comparison with unadjusted finite element models.

The specimen is a 30 ft x 7 ft composite slab supported by open-web steel joists. The slab overall thickness is 6 in. and the deck is 1 1/2 in. deep. Images of the specimen are shown in Figure 4.26. Pairs of angle bridging were used to provide lateral restraint of the top and bottom chords during construction and to hold the joists into position for deck placement. The steel deck was connected to the top chord using stand-off screws and the joists were connected to CMU walls using bottom chord bearing plates. The 8 in. lightweight CMU walls were reinforced with vertical and horizontal bars at spacings typical for this type of construction. The walls were constructed with the first mortar joint directly on the laboratory slab-on-grade, separated by a bond-breaking layer of plastic. Steel angles were connected to the walls as shown in Figure 4.26.

4.3.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker and accelerometer were placed at midspan. The measurements and predictions both indicate that the structure had one dominant mode that can be excited using one of the first four harmonics of the walking force. The measured and predicted natural frequencies for this natural mode are 7.75 Hz and 7.69 Hz (Predicted/Measured = 0.992), which practically match. The predicted mode shape is shown in Figure 4.27. A full modal sweep was not performed for this stage of the footbridge testing, so a measured mode shape is not available.

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping (0.34% of critical, determined using the log decrement method) was used in the model to allow a valid comparison. Constant hysteretic damping with stiffness proportional coefficient equal to 0.0068 (mass proportional coefficient equals zero) was used in the steady-state analysis. The measured and predicted accelerance FRF magnitudes are shown in Figure 4.28. The measured peak magnitude was 2.48 %g/lbf whereas the predicted was 1.96 %g/lbf (Predicted/Measured = 0.79).

Figure 4.26: Square End Joist Footbridge Pictures

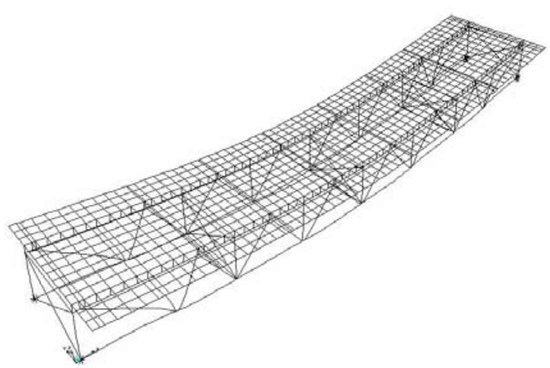


Figure 4.27: Predicted Mode Shape, Square-End Joist Footbridge

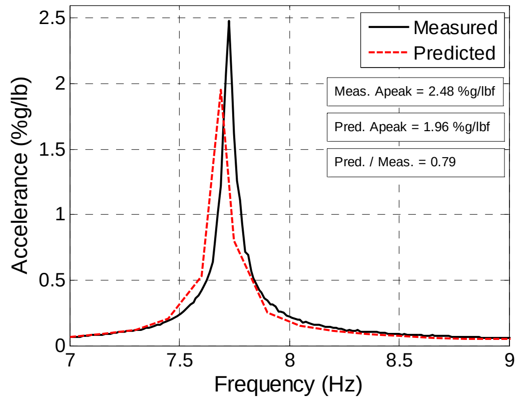


Figure 4.28: Measured and Predicted Accelerance Magnitudes, Square End Joist Footbridge

4.3.2 Response to Walking (Predictions Using Individual Footsteps)

Response to walking were measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. For this specimen, however, multiple trips were made back and forth during each measurement. The step frequency was 1.94 Hz (116 bpm) and 1.92 Hz (115 bpm) during tests and response history analyses, respectively, to cause resonance with the fourth harmonic of the walking force both in reality and in the model. Measured viscous modal damping was used in the model.

The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.29. Note that the beginning of the record shows some of the decaying response from the previous trip across the specimen. The measured acceleration spectrum indicates that the fourth harmonic of the walking force caused resonance. The measured acceleration waveform also indicates a fourth harmonic resonant build-up. The measured and predicted peak accelerations are 7.03%g and 7.01%g, respectively (Predicted / Measured = 1.00).

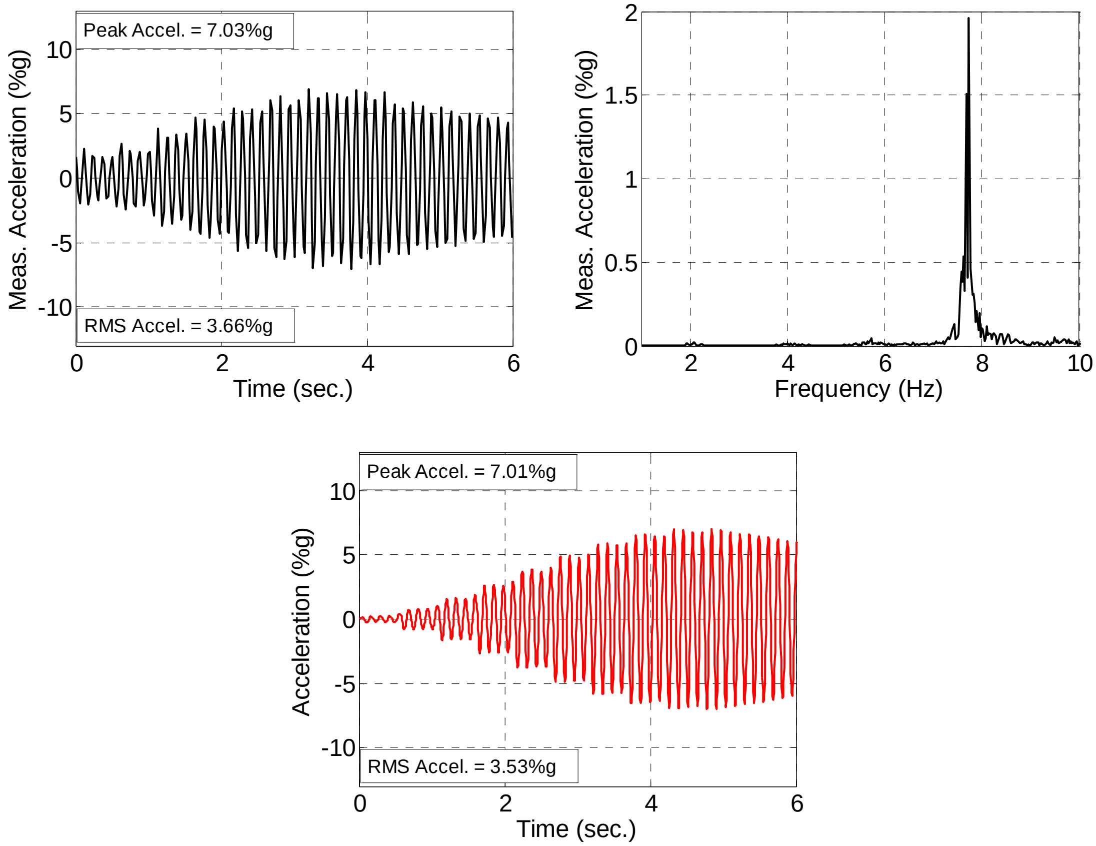


Figure 4.29: Acceleration Response to Walking, Individual Footsteps, Square End Joist Footbridge. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

4.3.3 Response to Walking (Predictions Using Fourier Series Loading)

Response to walking was also predicted using the Fourier series load as described in Chapter 3. As was the case for the prediction using individual footsteps, the step frequency was defined so that the fourth harmonic caused resonance during response history analysis. The measured walking acceleration waveform for the test with maximum response and the predicted waveform are shown in Figure 4.30. The measured and predicted peak accelerations are 7.03%g and 11.3%g, respectively (Predicted / Measured = 1.61).

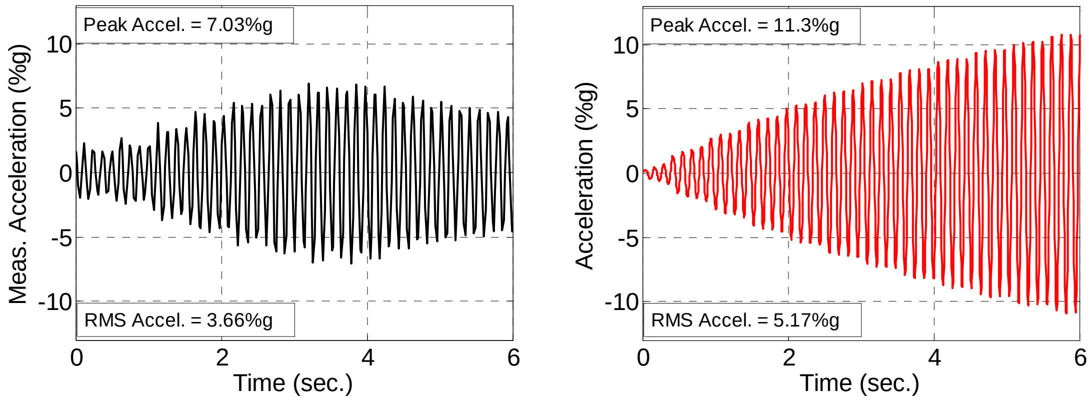


Figure 4.30: Acceleration Response to Walking in Bay 1, Fourier Series Loading, Square End Joist Footbridge. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

Figure 4.31 shows the response history prediction if only the fourth harmonic sinusoidal load is applied. The predicted peak acceleration, only including this term is 11.1%g, compared with 11.3%g predicted using all four terms (11.3/11.1=1.02). The other three harmonics provide a negligible portion of the predicted response in this case. The ratio of predicted-to-measured peak acceleration using only one term is 1.58.

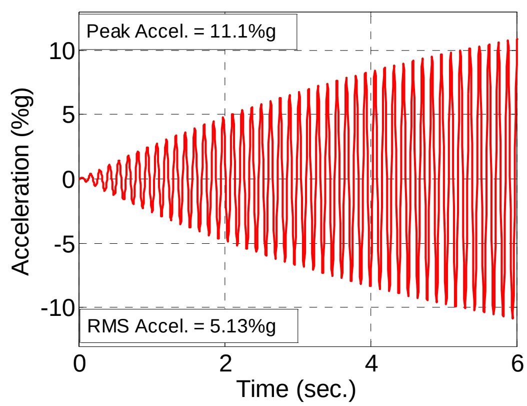


Figure 4.31: Acceleration Response to Walking, Fourier Series (Fourth Harmonic Force Only), Square End Joist Footbridge

4.3.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. Using this method, the acceleration response to walking is computed using the accelerance FRF magnitude. In general, the predicted

FRF magnitude will be available, but the predicted one will not. However, for the purposes of this research, both are used to predict the acceleration response to walking.

The predicted accelerance peak magnitude is 1.96 %g/lbf at a frequency of 7.69 Hz. The walking path was 30 ft and the damping ratio was 0.34% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 11.0%g. The measured peak acceleration was 7.03%g. The ratio of predicted-to-measured acceleration is 1.56, indicating a significant over-prediction.

The measured accelerance peak magnitude is 2.48 %g/lbf at a frequency of 7.75 Hz. The acceleration is predicted to be 14.0%g. The ratio of predicted-to-measured acceleration is 1.99, indicating a very significant over-prediction.

4.4 Shear-Connected Joist Footbridge

A three span joist footbridge was constructed and vibration tested at the Virginia Tech Structures and Materials Laboratory in 2006. The goal of the research was to determine if joists with shear connections have less severe vibration response than joists with typical joist seats. The specimen is a 90 ft x 7 ft composite slab supported by open-web steel joists. The slab overall thickness is 6 in. and the deck is 1 1/2 in. deep. Images of the specimen are shown in Figure 4.32. Pairs of angle bridging were used to provide lateral restraint of the top and bottom chords during construction and to hold the joists into position for deck placement. The steel deck was connected to the top chord using stand-off screws and the joists were connected to girders using shear connections. The girders were supported on W12 stub columns connected to the slab-on-grade using base plates on non-shrink grout.

4.4.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker was placed at the middle of the three bays to ensure that all single-curvature modes in the frequency range of interest for floor vibrations were excited. Accelerations were measured along the centerline of the specimen at quarter points and over supports. Three modes were measured and predicted in the frequency bandwidth of interest for floor vibrations, corresponding to the first three modes of a 3-span continuous beam. The natural frequencies are summarized in Table 4.6. The measured and predicted mode shapes are shown in Figure 4.33 through Figure 4.35. The measured and predicted natural frequencies and mode shapes are in excellent agreement.

Figure 4.32: Shear Connected Joist Footbridge Pictures

Figure 4.36 shows starburst plots for the first three modes, indicating that the first two are quasi-real and the third contains a little more complexity. It should be noted, however, that the DOF with mode shape vectors significantly out of phase with the others have relatively little response. Significant force glitches existed during these tests, so the apparent complexity might be attributed to errors in the experimental data.

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping was used in the model to allow a valid comparison. EMA FRF curve-fitting was used to determine the viscous modal damping ratios shown in Table 4.7. The curve-fit driving point FRFs are shown in Figure 4.37, indicating a very precise match of the parametric model and estimated FRFs.

Table 4.7 lists the damping ratios for the first three modes, indicating that the structure is very lightly damped. Figure 4.38 shows the measured and predicted driving point accelerance FRF magnitudes for the shaker in an end bay and in the center bay. The FRFs are similar, but the measured natural frequencies are a little closer together than are the predicted ones. Table 4.8 summarizes the FRF peak magnitudes. The model accurately predicted the first mode's peak driving point accelerance in each bay and reasonably predicted the peak magnitude of other modes except for the third mode in the end bay.

Table 4.6: Measured and Predicted Natural Frequencies, Shear Connected Joist Footbridge

	Natural Frequency		
Mode	Measured (Hz)	Predicted (Hz)	Predicted / Measured
1	7.78	7.70	0.990
2	8.08	8.18	1.01
3	8.78	9.19	1.05

Table 4.7: Damping Ratios, Shear Connected Joist Footbridge

Description	Damping (% of Critical)
Mode 1	0.17
Mode 2	0.27
Mode 3	0.25

Table 4.8: Measured and Predicted Accelerance Peak Magnitudes, Shear Connected Joist Footbridge

Description	Accelerance Peak Magnitude		
	Measured (%g/lbf)	Predicted (%g/lbf)	Pred. / Meas.
End Bay, Mode 1	1.46	1.57	1.08
End Bay, Mode 2	2.03	1.35	0.667
End Bay, Mode 3	0.313	0.516	1.65
Middle Bay, Mode 1	1.37	1.42	1.04
Middle Bay, Mode 3	1.55	2.08	1.34

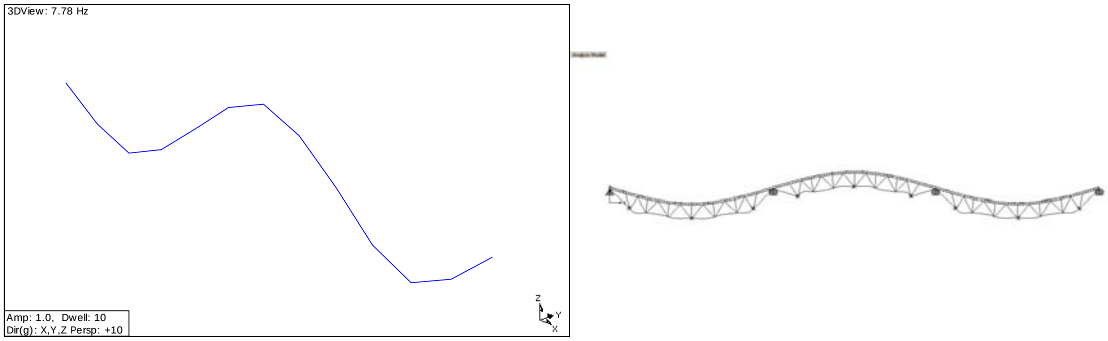


Figure 4.33: Mode 1, Shear Connected Joist Footbridge. (a) Measured Mode Shape (7.78 Hz); (b) Predicted Mode Shape (7.70 Hz)

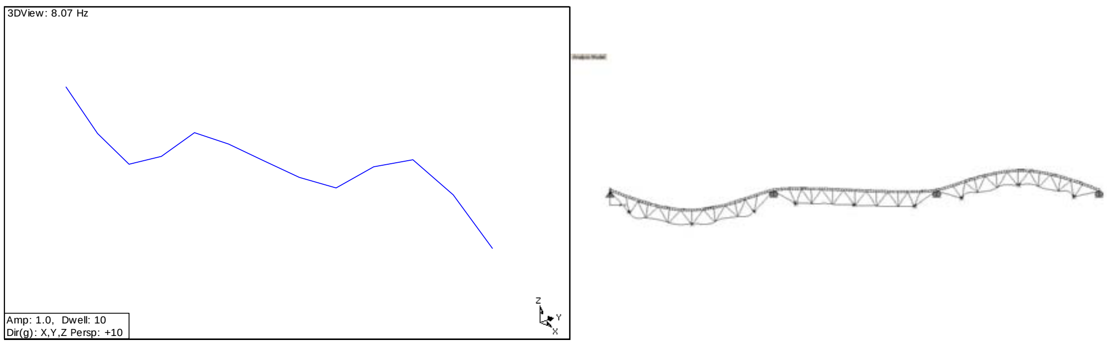


Figure 4.34: Mode 2, Shear Connected Joist Footbridge. (a) Measured Mode Shape (8.08 Hz); (b) Predicted Mode Shape (8.18 Hz)

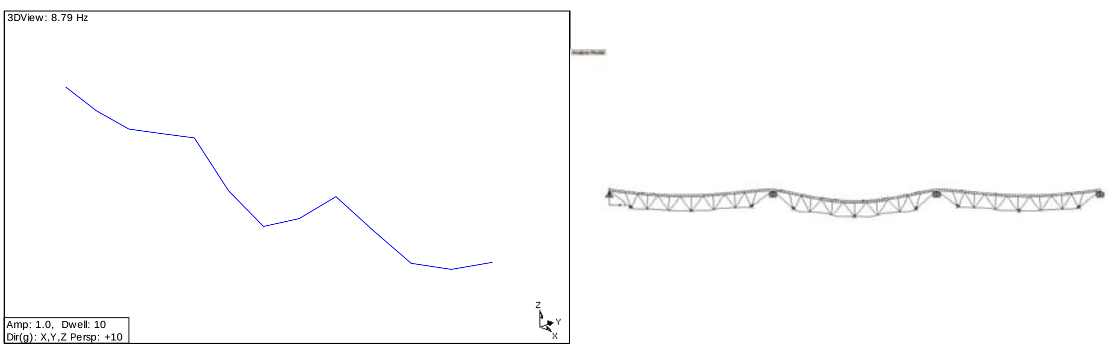


Figure 4.35: Mode 3, Shear Connected Joist Footbridge. (a) Measured Mode Shape (8.78 Hz); (b) Predicted Mode Shape (9.19 Hz)

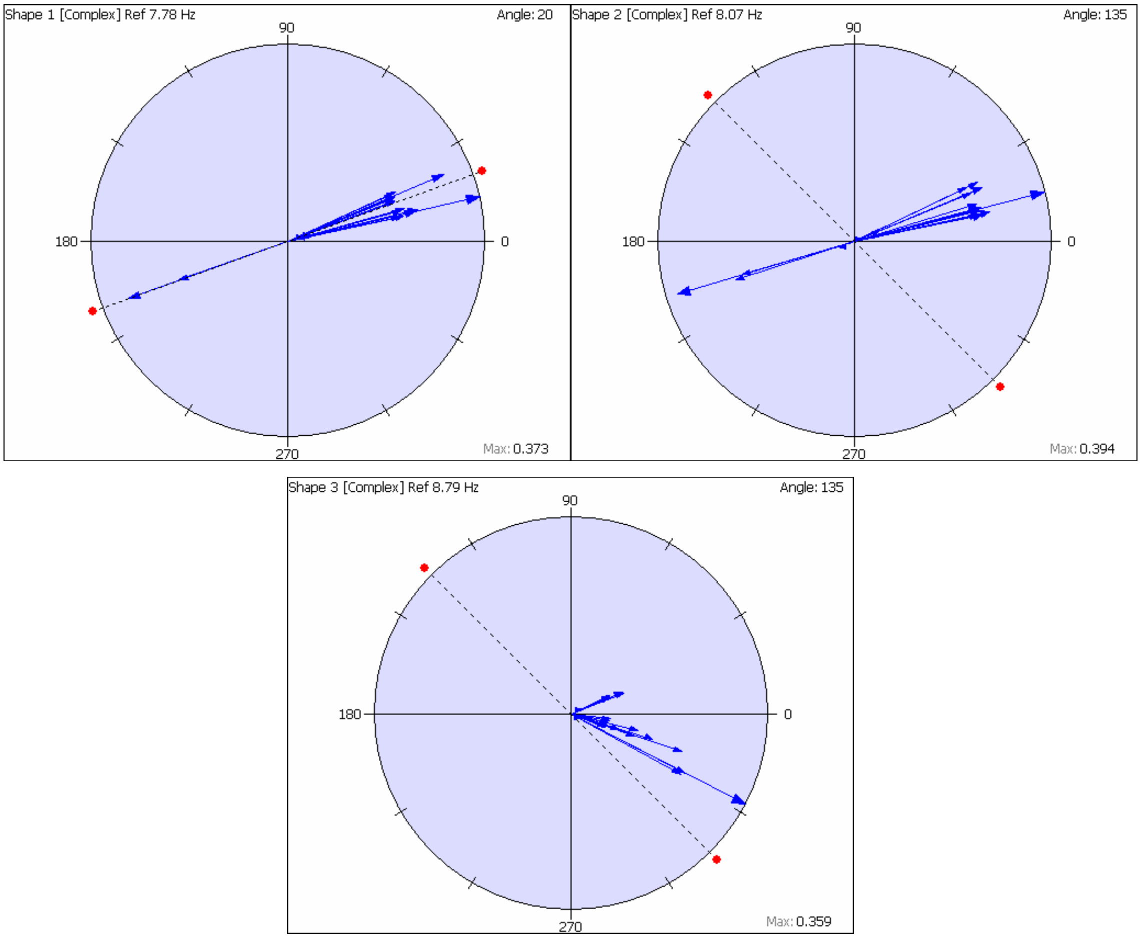


Figure 4.36: Starburst Plots for Shear Connected Joist Footbridge

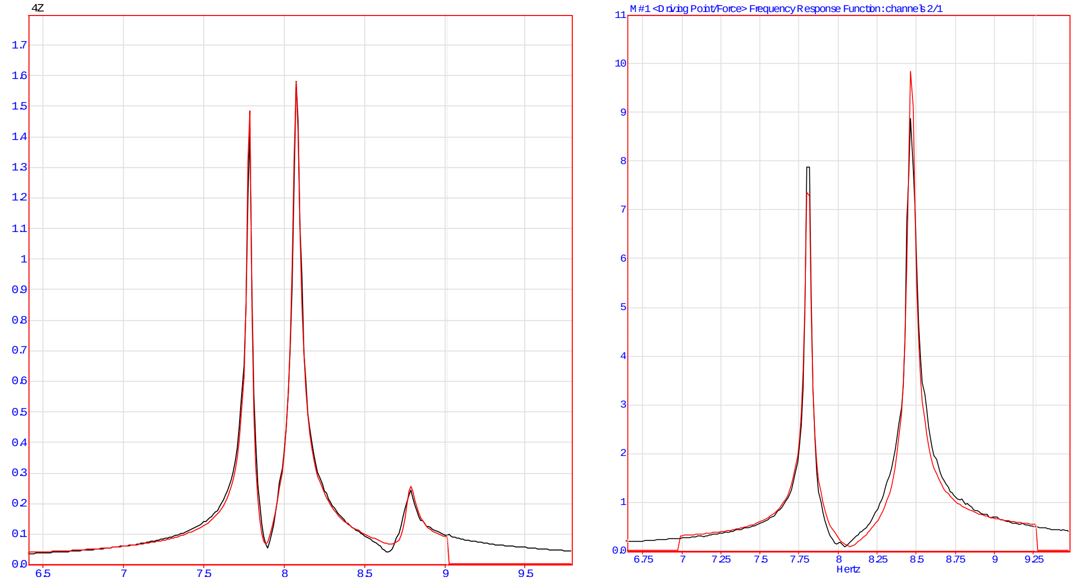


Figure 4.37: Curve-Fit FRFs for Shear Connected Joist Footbridge

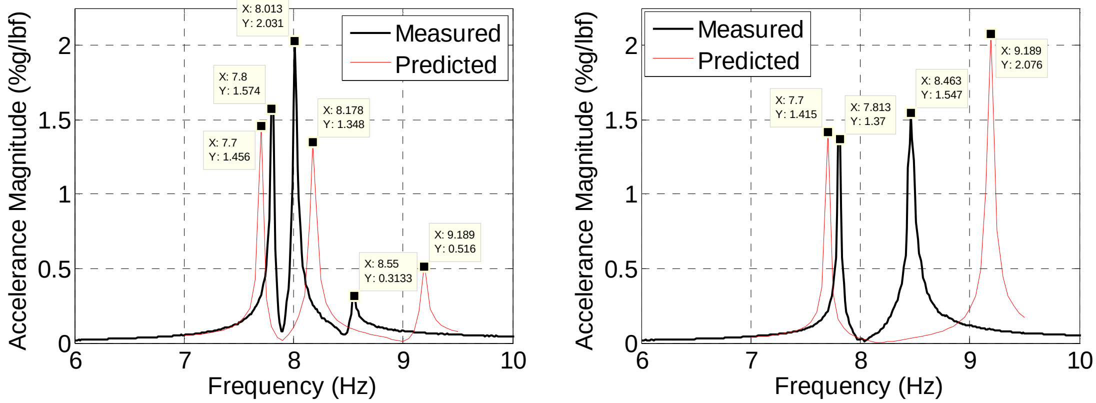


Figure 4.38: Measured and Predicted Accelerance Magnitudes, Shear Connected Joist Footbridge. (a) Center of Span 1; (b) Center of Span 2

4.4.2 Response to Walking (Prediction Using Individual Footsteps)

Response to walking in an end bay and in the middle bay were measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. The step frequency in each bay was the fourth subharmonic of the natural frequency to be excited, both in reality and in the model. Tests were performed with the intention of exciting Modes 1 and 2 in the end bay and Modes 1 and 3 in the middle bay—all four cases are reported in this section. Walking tests were performed at numerous other step frequencies ranging between 1.8 Hz (110 bpm) and 2.25 Hz (135 bpm), but these resulted in lower responses than walking at subharmonics of natural frequencies. Measured viscous modal damping was used in the model. Figure 4.39 through Figure 4.42 show the measured acceleration waveforms and spectra for the tests resulting in the maximum response. The predicted waveforms are also shown for comparison. Table 4.9 summarizes the measured and predicted peak accelerations. The model very accurately predicted the peak acceleration in the end bay. The peak acceleration during middle bay tests were slightly over-predicted and very significantly over-predicted for excitation of Mode 1 and 3, respectively. The latter was over-predicted because of inaccurate third mode property prediction as indicated in Figure 4.38(b).

Table 4.9 Measured and Predicted Peak Accelerations, Shear Connected Joist Footbridge, Individual Footstep Loading

Description	Peak Acceleration		
	Measured (%g)	Predicted (%g)	Predicted / Measured
End Bay, Mode 1	3.32	3.06	0.922
End Bay, Mode 2	4.14	4.24	1.02
Middle Bay, Mode 1	2.64	3.43	1.30
Middle Bay, Mode 3	4.67	10.2	2.18

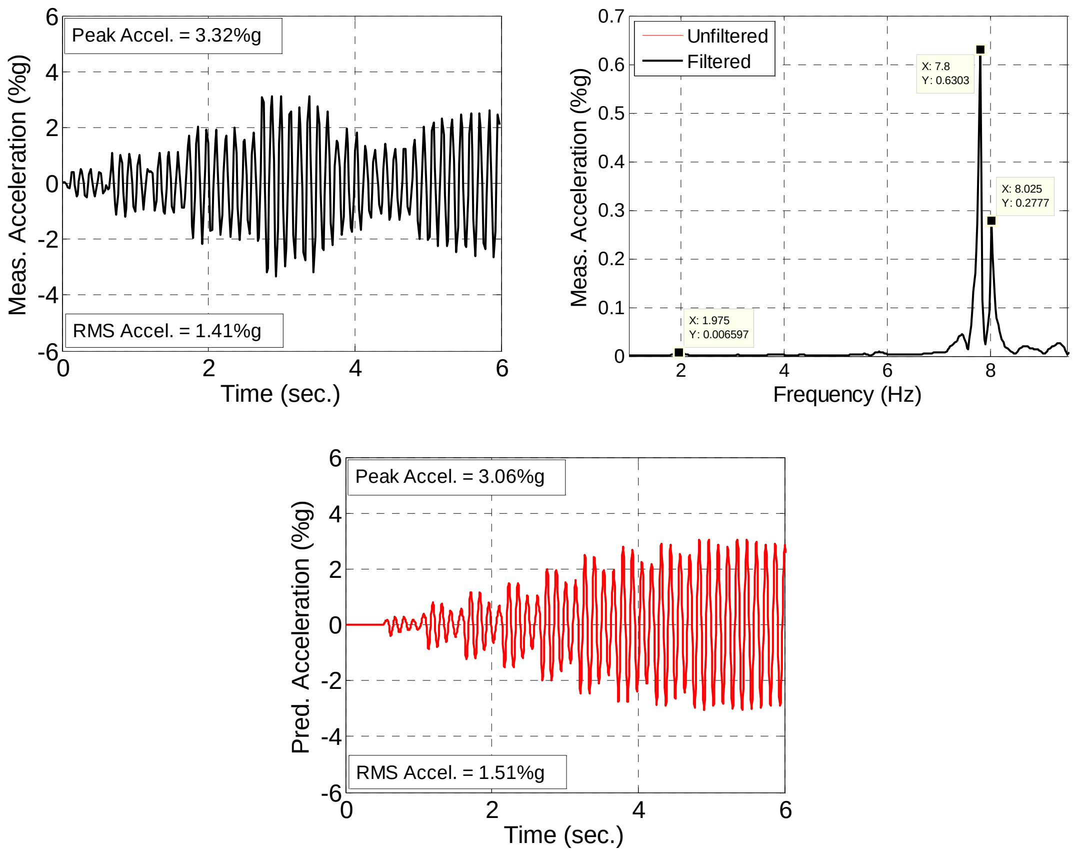


Figure 4.39: Measured and Predicted Accelerations Due to Walking, Shear Connected Joist Footbridge, End Bay, Mode 1 Excited. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

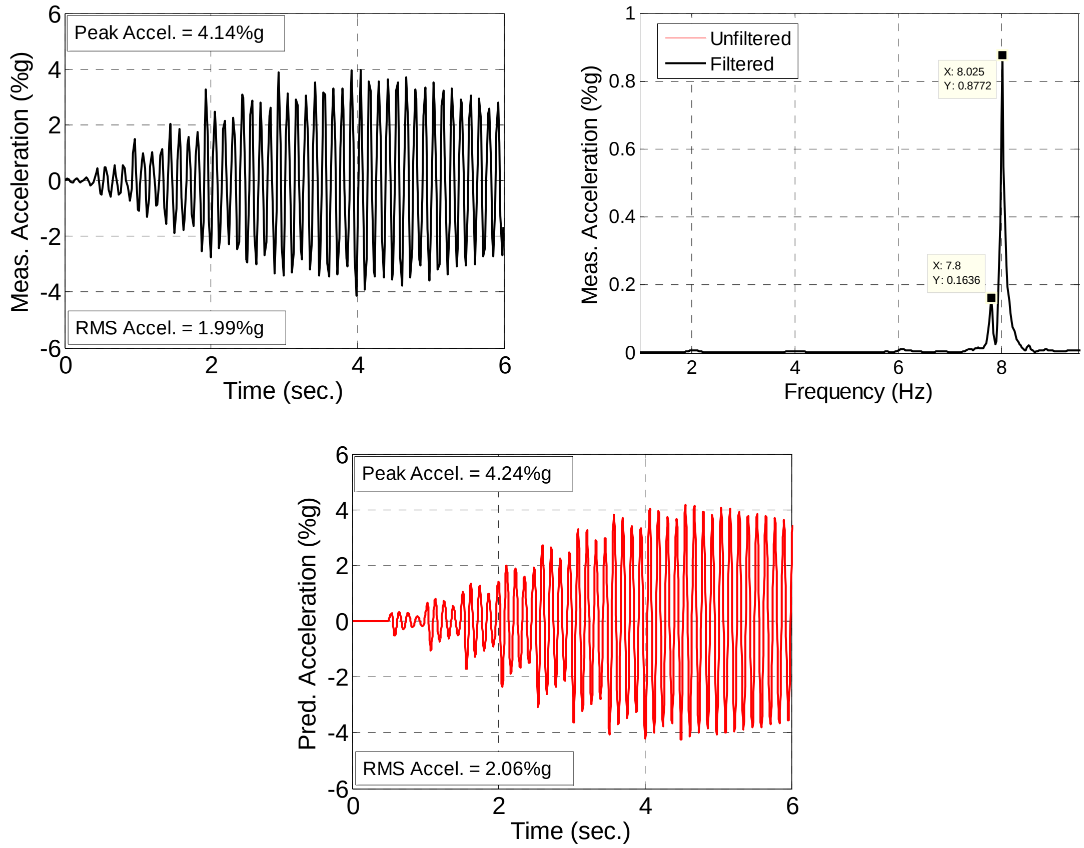


Figure 4.40: Measured and Predicted Accelerations Due to Walking, Shear Connected Joist Footbridge, End Bay, Mode 2 Excited. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

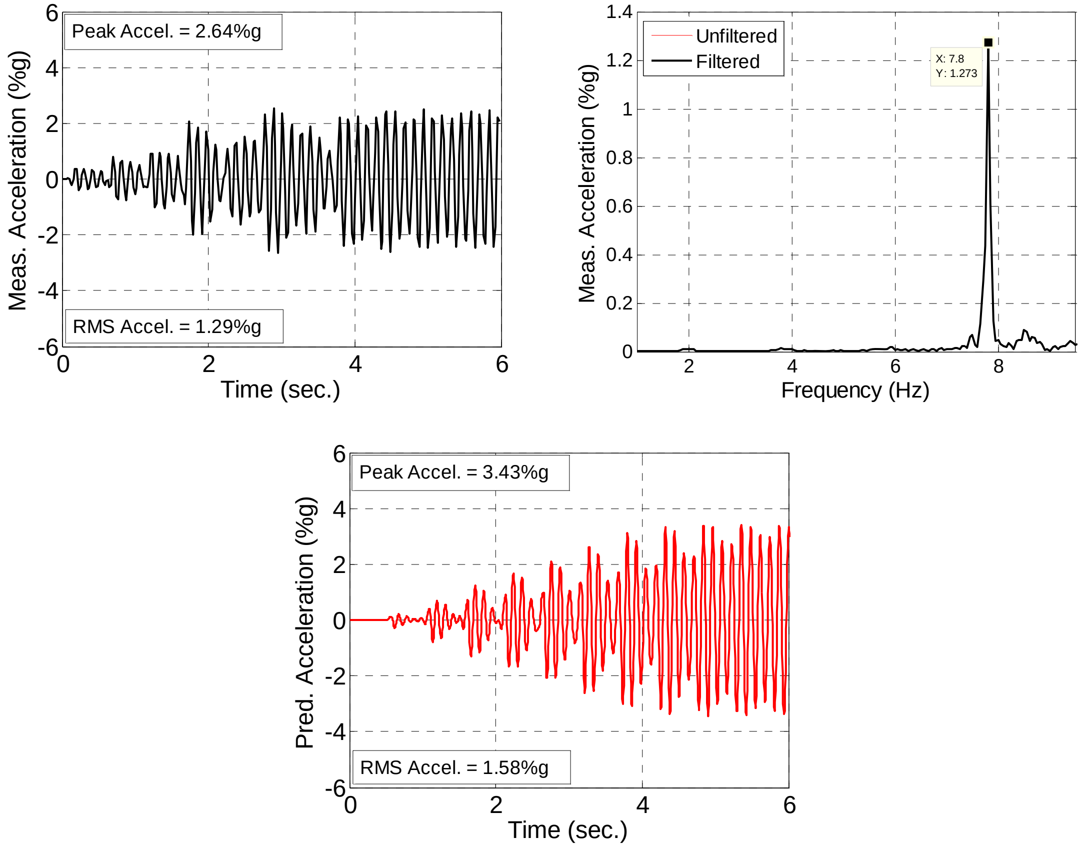


Figure 4.41: Measured and Predicted Accelerations Due to Walking, Shear Connected Joist Footbridge, Middle Bay, Mode 1 Excited. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

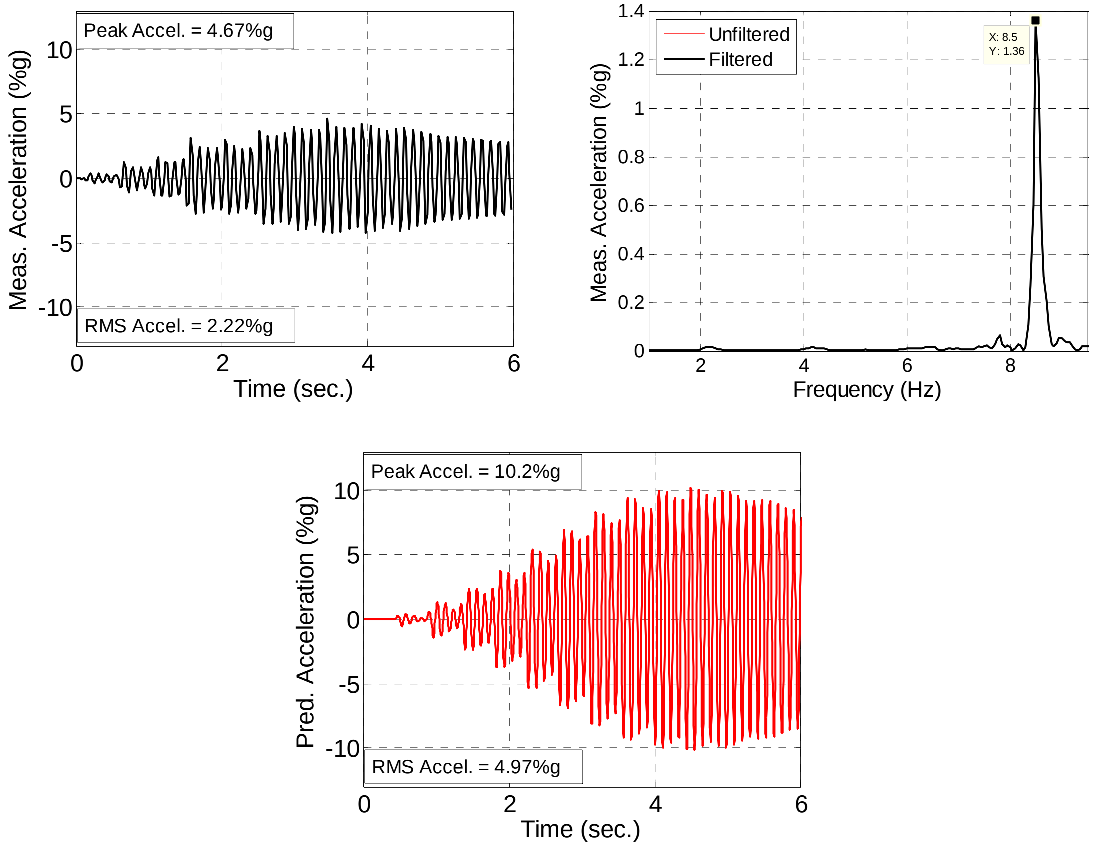


Figure 4.42: Measured and Predicted Accelerations Due to Walking, Shear Connected Joist Footbridge, Middle Bay, Mode 3 Excited. (a) Measured Acceleration Waveform; (b) Measured Acceleration Spectrum; (c) Predicted Acceleration Waveform

4.4.3 Response to Walking (Prediction Using Fourier Series Loading)

Response to walking in the end bay and middle bay were also predicted using the Fourier series load as described in Chapter 3. As was the case for the prediction using individual footsteps, the step frequency was defined so that the fourth harmonic caused resonance during response history analysis. Analyses were performed using all four terms of the Fourier series and using only the harmonic that matches the natural frequency. The following paragraphs present the results which are summarized in Table 4.10.

Table 4.10: Measured and Predicted Peak Accelerations, Shear Connected Joist Footbridge, Fourier Series Loading

Description	Peak Acceleration		
	Measured (%g)	Predicted (%g)	Predicted / Measured
End Bay, Mode 1, 4 Terms	3.32	5.42	1.63
End Bay, Mode 1, 1 Term	3.32	5.22	1.57
End Bay, Mode 2, 4 Terms	4.14	6.94	1.68
End Bay, Mode 2, 1 Term	4.14	6.68	1.61
Middle Bay, Mode 1, 4 Terms	2.64	5.30	2.01
Middle Bay, Mode 1, 1 Term	2.64	5.15	1.95
Middle Bay, Mode 3, 4 Terms	4.67	11.4	2.44
Middle Bay, Mode 3, 1 Term	4.67	11.0	2.36

Figure 4.43 and Figure 4.45 show the comparison of measured and predicted waveforms for walking in the first bay exciting the first and second natural modes, considering all four terms of the Fourier series. The measured waveforms show weak fourth harmonic resonant build-ups, most likely due to the combination of imperfect step frequency and relatively closely spaced modes. Both predictions were high by an average of approximately 65%, some of which is attributable to the load being applied at mid-bay during the entire response history analysis. The waveforms also indicate a reasonable prediction to 3-4 sec. at which time the measured waveform stops increasing whereas the predicted one continues to increase. Figure 4.44 and Figure 4.46 show the response predictions in the end bay if only the fourth Fourier series term is included. These indicate that the inclusion of the other three terms have a very small effect.

Figure 4.47 and Figure 4.49 show the comparison of measured and predicted waveforms for walking in the middle bay exciting the first and third natural modes, considering all four terms of the Fourier series. Figure 4.47 shows almost no resonant build-up and Figure 4.49 shows a very weak one. Both predictions were high by an average of approximately 122%, some of which is attributable to the load being applied at mid-bay. Figure 4.48 and Figure 4.50 show the response predictions in the middle bay if only the fourth Fourier series term is included. These indicate that the inclusion of the other three terms have a very small effect.

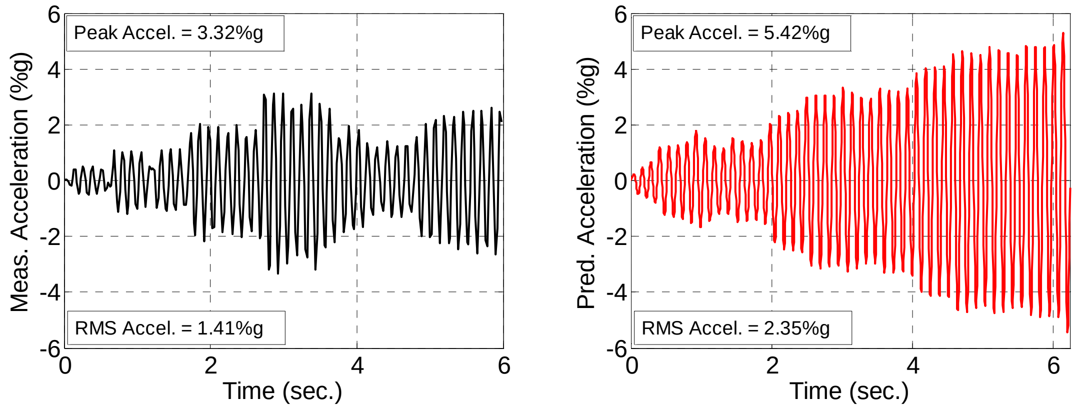


Figure 4.43: Acceleration Response to Walking in End Bay, Mode 1 Excited, Fourier Series Loading, Shear Connected Joist Footbridge. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

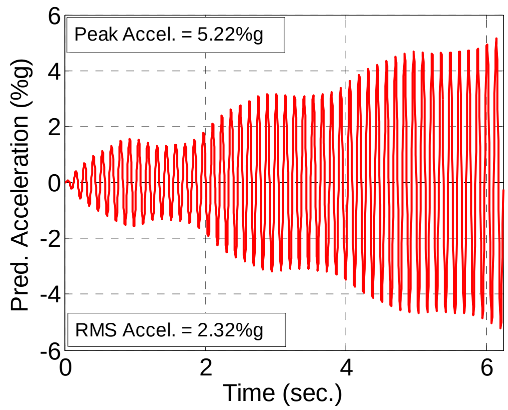


Figure 4.44: Acceleration Response to Walking in End Bay, Mode 1 Excited, Fourier Series (Fourth Harmonic Force Only), Shear Connected Joist Footbridge

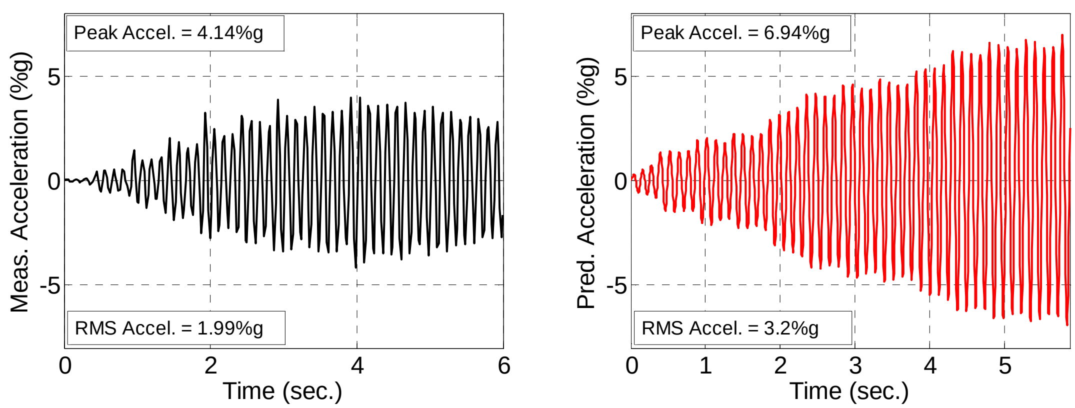


Figure 4.45: Acceleration Response to Walking in End Bay, Mode 2 Excited, Fourier Series Loading, Shear Connected Joist Footbridge. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

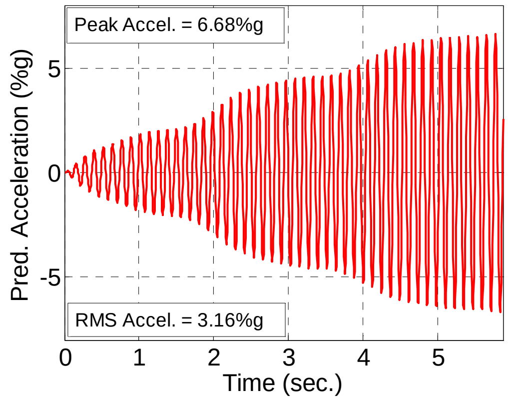


Figure 4.46: Acceleration Response to Walking in End Bay, Mode 2 Excited, Fourier Series (Fourth Harmonic Force Only), Shear Connected Joist Footbridge

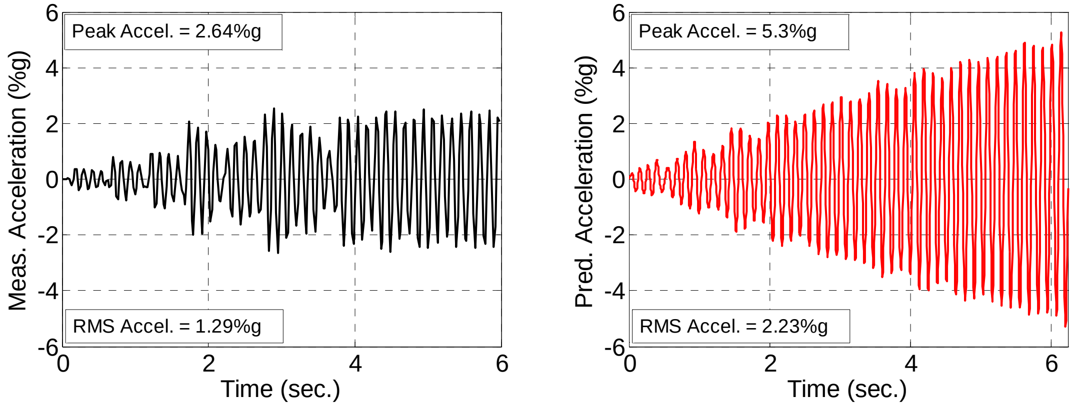


Figure 4.47: Acceleration Response to Walking in Middle Bay, Mode 1 Excited, Fourier Series Loading, Shear Connected Joist Footbridge. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

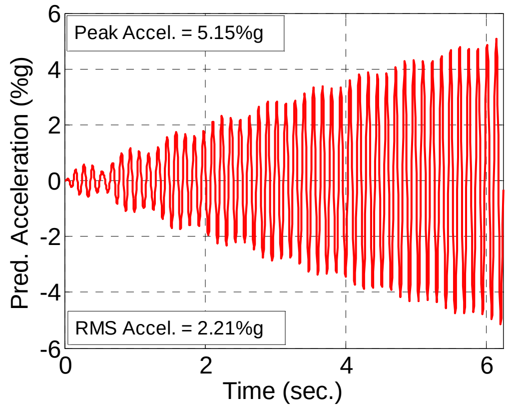


Figure 4.48: Acceleration Response to Walking in Middle Bay, Mode 1 Excited, Fourier Series (Fourth Harmonic Force Only), Shear Connected Joist Footbridge

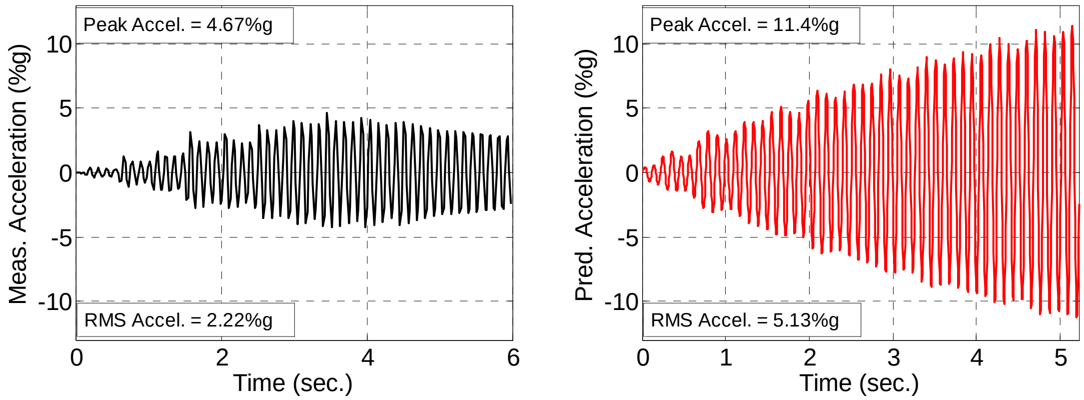


Figure 4.49: Acceleration Response to Walking in Middle Bay, Mode 3 Excited, Fourier Series Loading, Shear Connected Joist Footbridge. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform

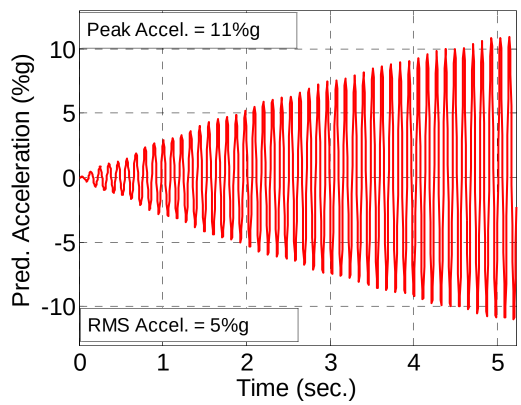


Figure 4.50: Acceleration Response to Walking in Middle Bay, Mode 3 Excited, Fourier Series (Fourth Harmonic Force Only), Shear Connected Joist Footbridge

4.4.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. The response was predicted using the predicted FRF magnitude and the measured FRF magnitude. The first two bending modes provide significant response for excitation in the end bay whereas the first and third bending modes provide response in the middle bay. Each mode is used separately to predict the response. This specimen was nearly unique among the studied floors because its measured and predicted modes were in excellent agreement, including mode shapes and order. This allowed a mode-by-mode comparison of acceleration responses to walking.

In the end bay, the first mode predicted accelerance peak magnitude is 1.46 %g/lbf at a frequency of 7.7 Hz. The walking path was 30 ft and the damping ratio was

0.17% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 5.13%g whereas the measured peak acceleration due to walking was 3.32%g. The ratio of predicted-to-measured accelerations is 1.55.

In the end bay, the first mode measured accelerance peak magnitude is 1.57 %g/lbf at a frequency of 7.8 Hz. The acceleration is predicted to be 5.60%g. The ratio of predicted-to-measured accelerations is 1.69.

In the end bay, the second mode predicted accelerance peak magnitude is 1.35 %g/lbf at a frequency of 8.18 Hz. The damping ratio was 0.27% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 6.92%g whereas the measured peak acceleration due to walking was 4.14%g. The ratio of predicted-to-measured accelerations is 1.67.

In the end bay, the second mode measured accelerance peak magnitude is 2.03 %g/lbf at a frequency of 8.01 Hz. The acceleration is predicted to be 10.3%g. The ratio of predicted-to-measured accelerations is 2.49.

In the middle bay, the first mode predicted accelerance peak magnitude is 1.42 %g/lbf at a frequency of 7.7 Hz. The walking path was 30 ft long. Using the method described in Section 3.5, the acceleration is predicted to be 4.98%g whereas the measured peak acceleration due to walking was 2.64%g. The ratio of predicted-to-measured accelerations is 1.89.

In the middle bay, the first mode measured accelerance peak magnitude is 1.37 %g/lbf at a frequency of 7.81 Hz. The acceleration is predicted to be 4.88%g. The ratio of predicted-to-measured accelerations is 1.85.

In the middle bay, the third mode predicted accelerance peak magnitude is 2.08 %g/lbf at a frequency of 9.19 Hz. Using the method described in Section 3.5, the acceleration is predicted to be 11.2%g whereas the measured peak acceleration due to walking was 4.67%g. The ratio of predicted-to-measured accelerations is 2.40. This comparison is a very clear example of an accelerance peak magnitude error causes a significant over-prediction of acceleration due to walking.

In the middle bay, the third mode measured accelerance peak magnitude is 1.55 %g/lbf at a frequency of 8.46 Hz. The acceleration is predicted to be 7.77%g. The ratio of predicted-to-measured accelerations is 1.66. Note that the acceleration prediction is much better using the measured FRF peak magnitude in this case, although it is still significantly conservative. However, this over-prediction is much more similar to most of the other predictions included in this research using this method.

4.5 Riverside Medical Office Building

One floor of a building under construction was vibration tested in April 2006. The building is four stories, 55,000 sq. ft, and is shown in Figure 4.51. It was built in two stages, with the nearside being built at the time of testing, as shown in Figure 4.52.

The Second Floor (second elevated floor—ground floor is parking) was chosen because its topside supported less construction material than the other floors. The slab topside was mostly clear of construction material and the underside supported only very minimal piping and ductwork. See Figure 4.53 and Figure 4.54. Full height steel studs were installed only in the core area. The floor was a nearly bare slab.

The floor was built using a conventional steel framed system with composite slabs. The floor plan is shown in Figure 4.55. The normal weight concrete slab is 6 1/2 in. total thickness on 2 in. composite deck. Rigid frames exist along Gridlines B and D, creating bays without simply supported edges, and therefore an opportunity to test the proposed evaluation procedures on a building with significant irregularities. Most bays are 30 ft square.

Figure 4.51: Riverside MOB (Finished)

Figure 4.52: Riverside MOB (During Vibration Tests)

Figure 4.53: Riverside MOB Slab Topside Condition

Figure 4.54: Riverside MOB Slab Underside Condition

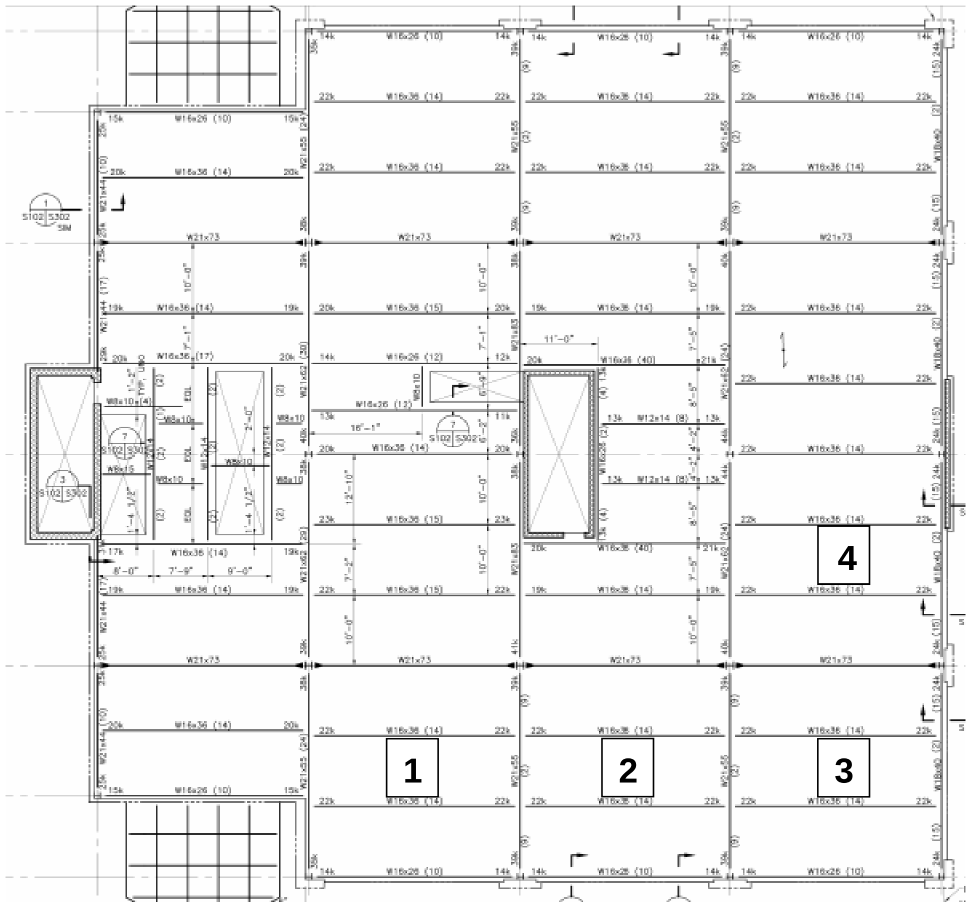


Figure 4.55: Riverside MOB Framing Plan

4.5.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker was placed in the four bays indicated in Figure 4.55. Accelerations were measured at the centers of bays and at the spandrel location closest to the driving point during each test. In the model, the spandrel was modeled using the recommendation by Barrett (2006): multiply the transformed moment of inertia by 2.5.

The measurements and predictions both indicate that the floor has several vibration modes that can be excited using one of the first four harmonics of the walking force. The first six measured natural frequencies and first nine predicted natural frequencies are between 6.4 Hz and 8.2 Hz. The measured and predicted mode shapes are shown in Figure 4.56 and Figure 4.57. Only three of the first six measured modes were predicted by the model. These are shown in Figure 4.58. Of these, the predicted natural frequencies were within approximately 10% of the measured ones. The other measured and predicted natural frequencies cannot be compared because the mode shapes are not similar.

Figure 4.59 shows starburst plots for the first six measured mode shapes. The following modes are quasi-real: Mode 1, Mode 3, and Mode 6. The other three modes are significantly more complex. It is interesting to note that the three quasi-real modes are associated with displacement of Bays 1 through 3, with little displacement of Bay 4 and the other bays along that edge of the building. The modes with more complexity are associated primarily with displacement of the bays along the east edge of the building. This indicates that damping is spatially distributed approximately the same as stiffness and mass in Bays 1 through 3 and that it is not distributed in this manner along the east edge of the building.

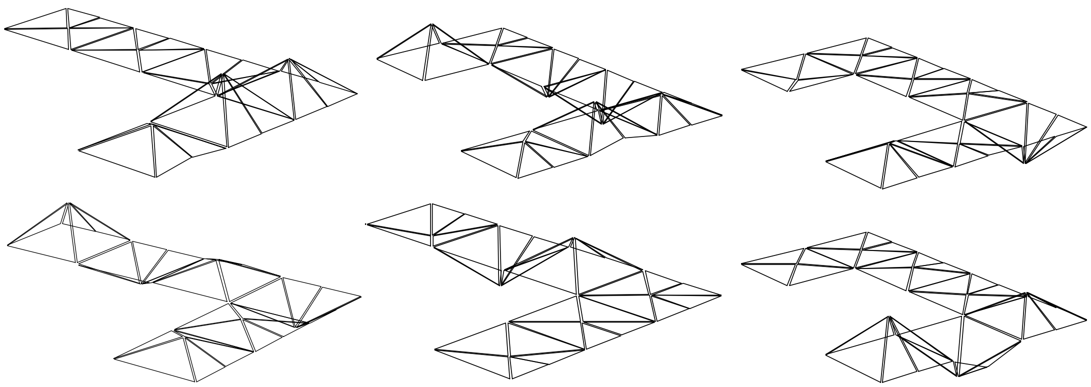


Figure 4.56: Riverside MOB Measured Mode Shapes (a) Mode 1: 6.42 Hz; (b) Mode 2: 6.61 Hz; (c) Mode 3: 7.01 Hz; (d) Mode 4: 7.14 Hz; (e) Mode 5: 7.46 Hz; (f) Mode 6: 8.14 Hz

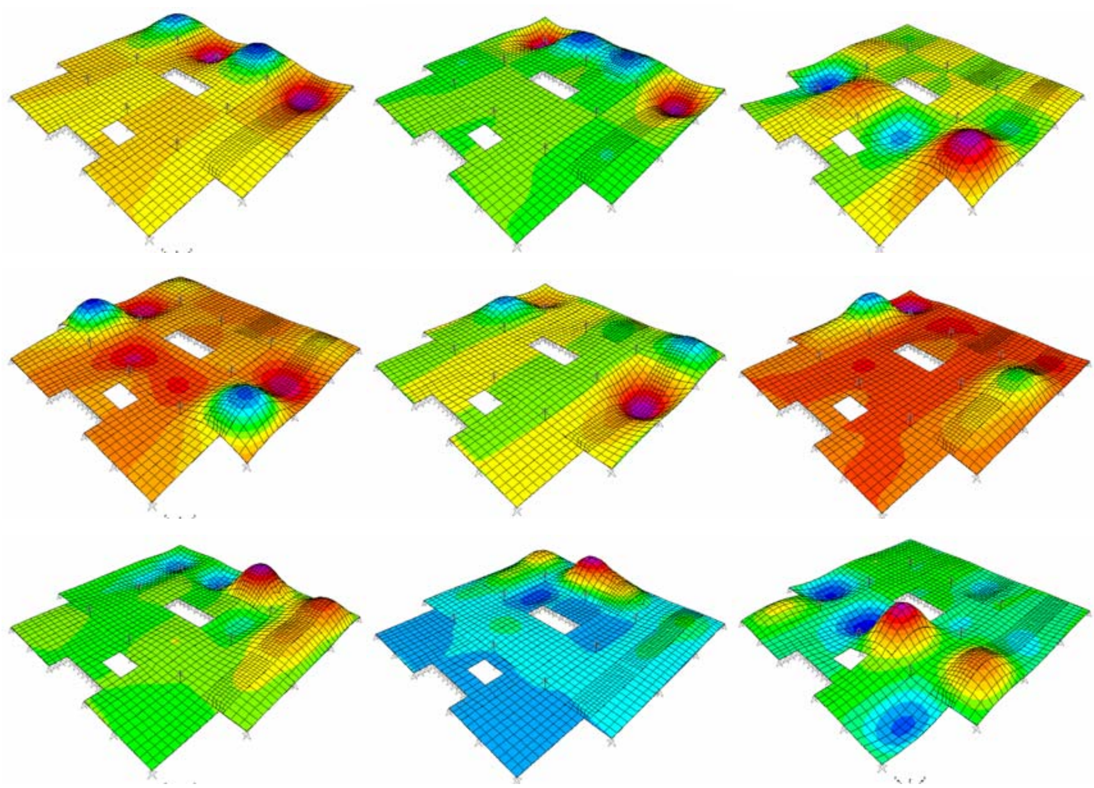
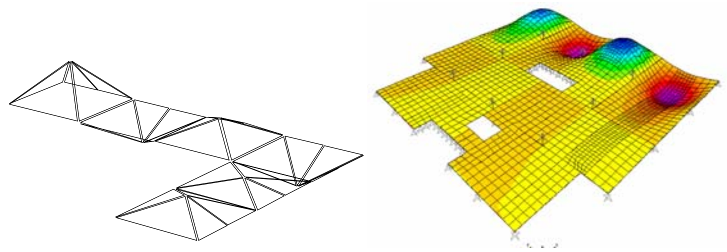
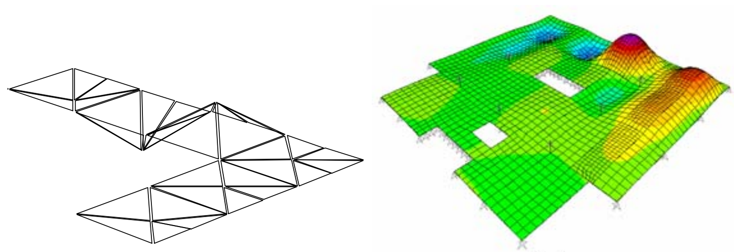


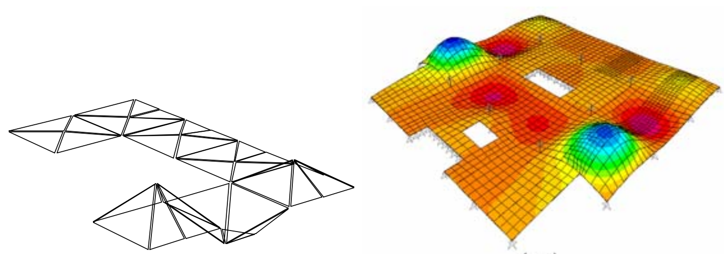
Figure 4.57: Riverside MOB Predicted Mode Shapes (a) Mode 1: 6.50 Hz; (b) Mode 2: 6.93 Hz; (c) Mode 3: 7.12 Hz; (d) Mode 4: 7.34 Hz; (e) Mode 5: 7.45 Hz; (f) Mode 6: 7.51 Hz; (g) Mode 7: 7.88 Hz; (h) Mode 8: 7.94 Hz; (i) Mode 9: 8.20 Hz



(a) Measured Mode 4 (7.14 Hz), Predicted Mode 1 (6.50 Hz)



(b) Measured Mode 5 (7.46 Hz), Predicted Mode 7 (7.88 Hz) Predicted



(c) Measured Mode 6 (8.14 Hz), Predicted Mode 4 (7.34 Hz)

Figure 4.58: Riverside MOB Comparable Measured and Predicted Mode Shapes

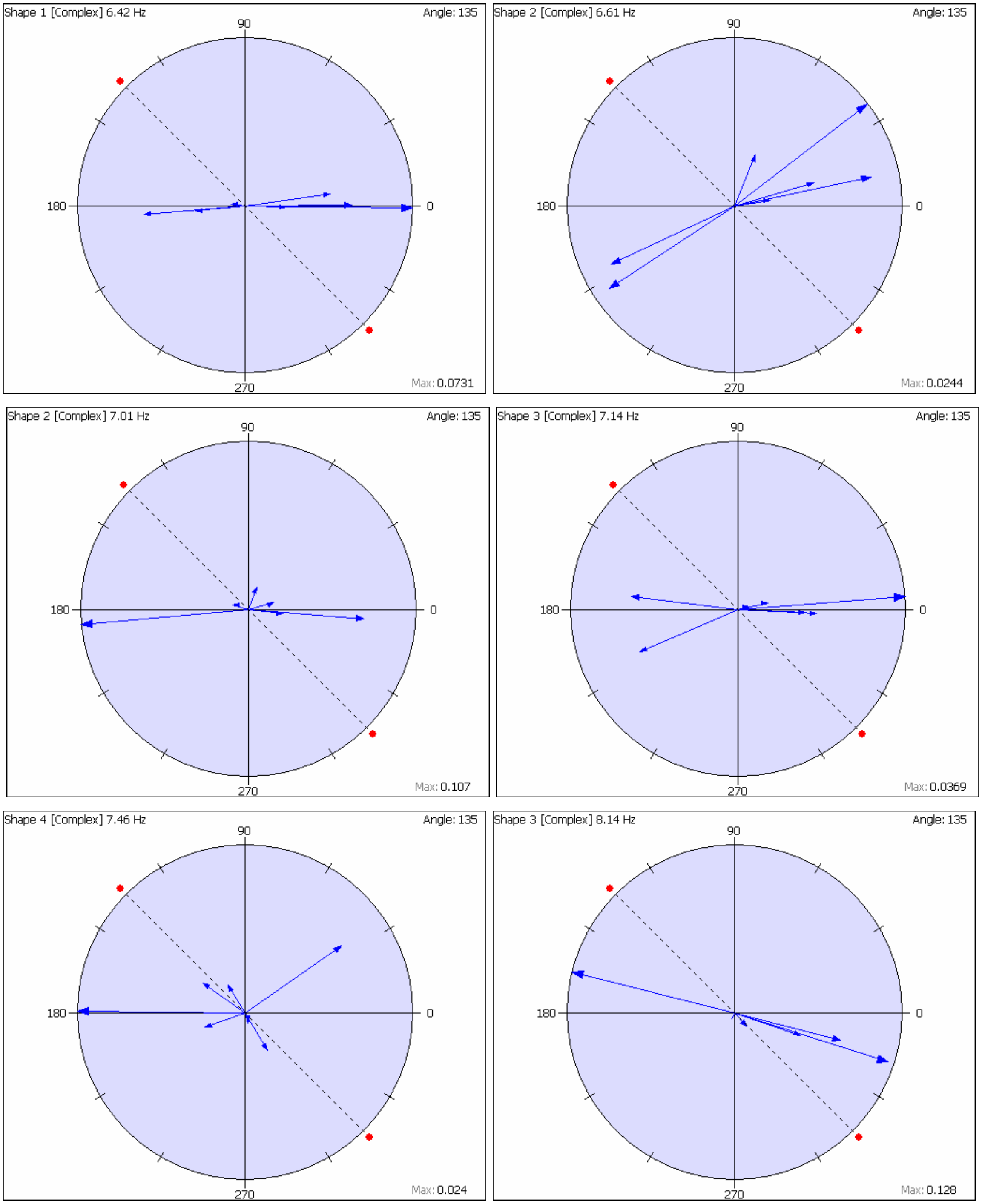


Figure 4.59: Starburst Plots, Riverside MOB

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping was used in the model to allow a valid comparison. EMA FRF curve-fitting was used to determine the viscous modal damping ratios shown in Table 4.11. Interestingly, the damping ratios are remarkably consistent for all of the measured modes, all being between 0.51% and 0.61% of critical. The curve-fit driving point FRFs for are all four bays are shown in Figure 4.60, indicating a very precise match of the parametric model and estimated FRFs. Constant hysteretic damping with mass proportional coefficient set equal to zero was used for the FRF predictions in the four bays. Stiffness proportional coefficients for the four bays were 0.0102, 0.0108, 0.0116, and 0.0108. These were chosen using judgment because the measured and predicted modes did not correspond in most cases, as discussed in a previous paragraph.

The measured and predicted driving point FRF magnitudes are shown in Figure 4.61, indicating only partial agreement between the measurements and predictions. The dominant mode peak magnitudes are summarized in Table 4.12. In Bay 1, the model accurately predicted the peak magnitude, but not the corresponding natural frequency. In Bay 2, the model very significantly over-predicted the peak magnitude because it predicted the 7.5 Hz mode (Figure 4.57(e)) which had significant displacement almost exclusively in Bay 2, therefore under-predicting the effective mass. In reality, this mode did not exist. In Bay 3, the model accurately predicted both the peak magnitude and frequency. This is primarily due to chance, however, because the measured (7.0 Hz, Figure 4.56(c)) and predicted (Figure 4.57(b)) modes are not of the same shape. The model inaccurately predicted the accelerance peak in Bay 4 because the predicted mode shape at 7.9 Hz (Figure 4.57(g)) over-estimated the displacement in Bay 4 compared to elsewhere on the floor, therefore under-predicting the effective mass. In reality, no mode shaped like Figure 4.57(g) existed.

Table 4.11: Damping Ratios, Riverside MOB

Measured Mode Number	Damping (% of Critical)
1	0.51
2	0.59
3	0.58
4	0.61
5	0.54
6	0.54

Table 4.12: Measured and Predicted Accelerance Peak Magnitudes, Riverside MOB

Bay Number	Accelerance Peak Magnitude		
	Measured (%g/lbf)	Predicted (%g/lbf)	Pred. / Meas.
1	0.151	0.153	1.01
2	0.128	0.236	1.84
3	0.118	0.116	0.983
4	0.0770	0.192	2.49

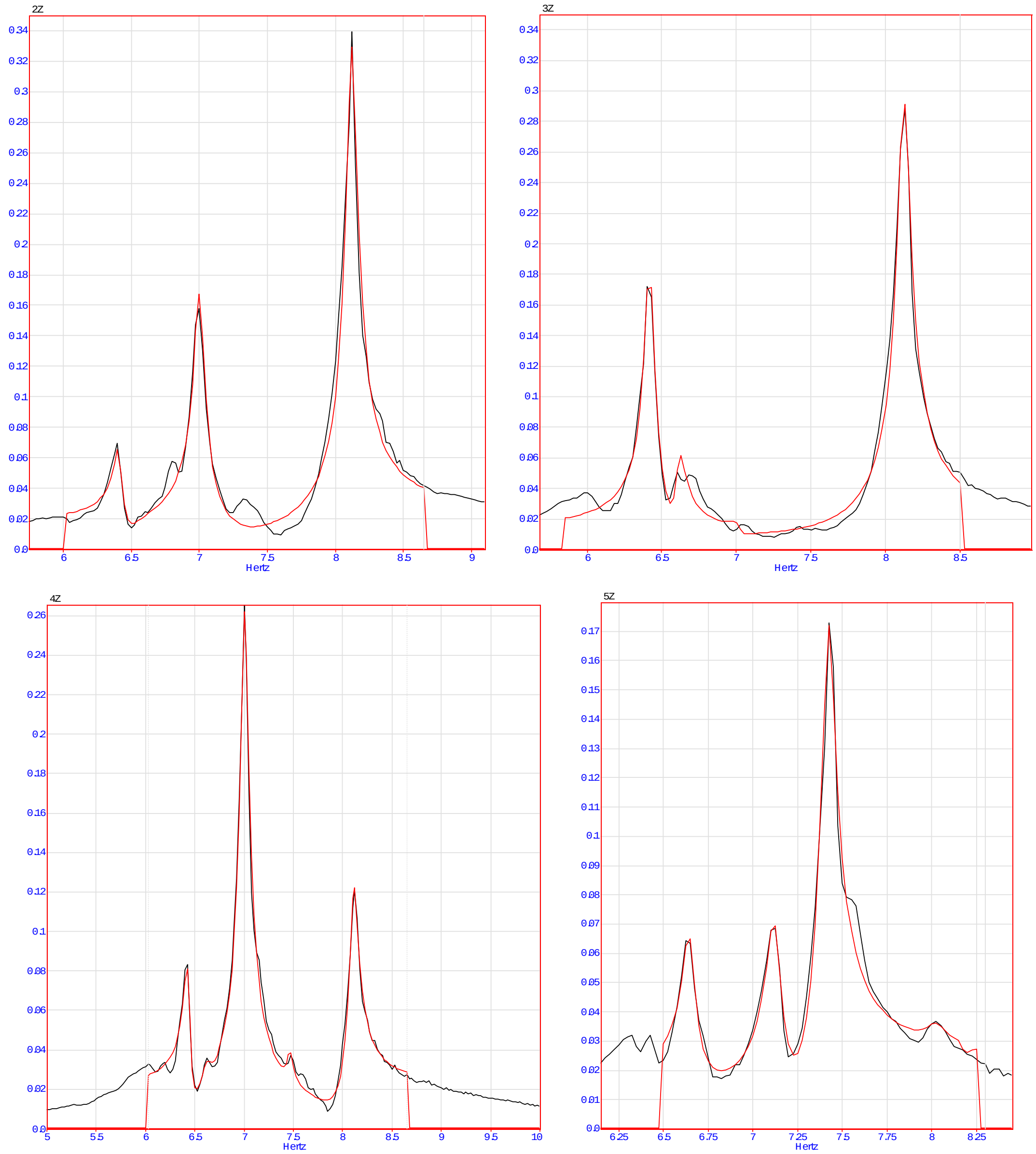


Figure 4.60: Curve-Fit Driving Point FRFs for Riverside MOB. (a) Bay 1; (b) Bay 2; (c) Bay 3; (d) Bay 4.

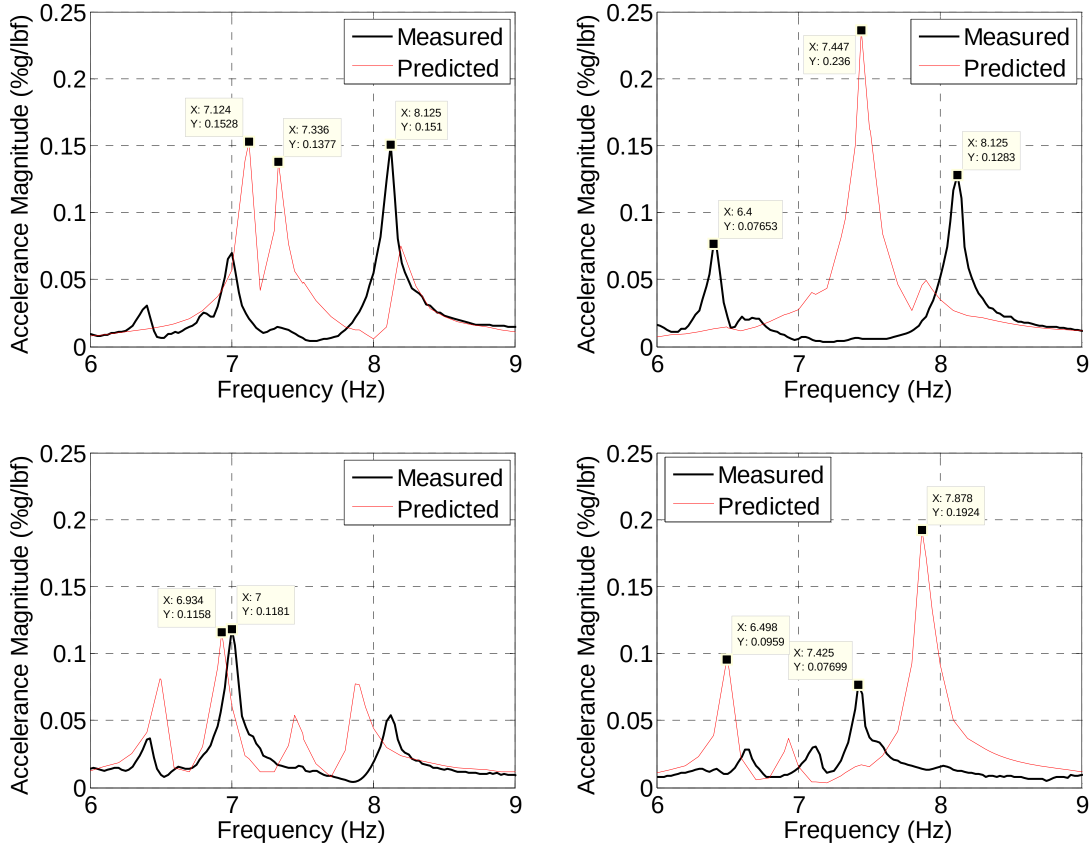


Figure 4.61: Measured and Predicted Accelerance FRF Magnitudes (a) Shaker in Bay 1; (b) Shaker in Bay 2; (c) Shaker in Bay 3; (d) Shaker in Bay 4

One additional model was created to investigate the viability of creating a model of only a portion of the floor. Measured and predicted natural frequencies, mode shapes, and accelerance magnitudes are compared. If a partial model predicts natural frequencies, mode shapes, and FRF magnitudes similar to those predicted by the full model, then accelerations due to walking will be similar using the partial model. Of course, use of a partial model would save significant time and effort spent modeling such floors. This floor is the only one studied in this research that is large enough to use for this investigation. The model was created of Bays 1 through 3, plus approximately one adjacent bay. The first five predicted mode shapes are shown in Figure 4.62. These mode shapes correspond to the following modes predicted using the full model: 1 (6.50 Hz), 3 (6.93 Hz), 5 (7.45 Hz), 7 (7.88 Hz), and 9 (8.20 Hz). The full model predicted several mode shapes not predicted by the partial model. The maximum difference between the natural frequencies predicted by the full model and partial model is 4%, so the corresponding predicted natural frequencies were not significantly affected by the use of a partial model.

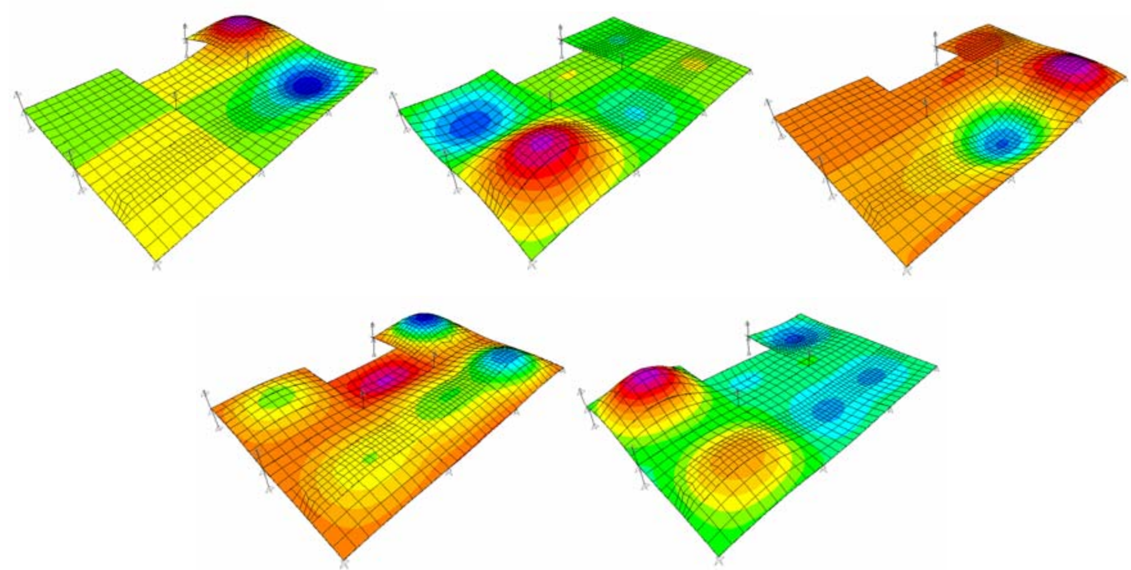


Figure 4.62: Riverside MOB Reduced Model Predicted Mode Shapes (a) Mode 1: 6.47 Hz; (b) Mode 2: 7.03 Hz; (c) Mode 3: 7.42 Hz; (d) Mode 4: 7.69 Hz; (e) Mode 5: 7.86 Hz.

Accelerance FRF magnitudes are also compared—see Figure 4.63. In Bay 1, the peak magnitude is reasonably predicted using the full model (Predicted / Measured = 1.01, but the natural frequencies are a bit different), but very significantly over-predicted using the partial model (Predicted / Measured = 1.97). In Bay 2, the full model and partial model very significantly over-predicted the accelerance, but the partial model was the least accurate of the two. Interestingly, in Bay 3, the full model accurately predicted the dominant mode's FRF peak magnitude at 7.03 Hz (measured), but the partial model did not predict a FRF peak near this frequency. As indicated throughout much of this research, the most severe errors in modal property prediction are FRF peak magnitudes very significantly over-predicted due to under-prediction of effective mass. These lead to very significant over-prediction of acceleration response to walking. It is also clear that floor motion several bays away plays a significant role in determining the effective mass in a given bay—see Figure 4.56(b) for example. Therefore, it should be no surprise that partial models will less accurately predict FRF peak magnitudes and therefore acceleration due to walking in some bays. While these comprise a small data set, the use of a partial model does not appear to be feasible for floors similar to the Riverside MOB second floor. It is anticipated, however, that much larger floor areas can be adequately modeled using fairly large partial floor models.

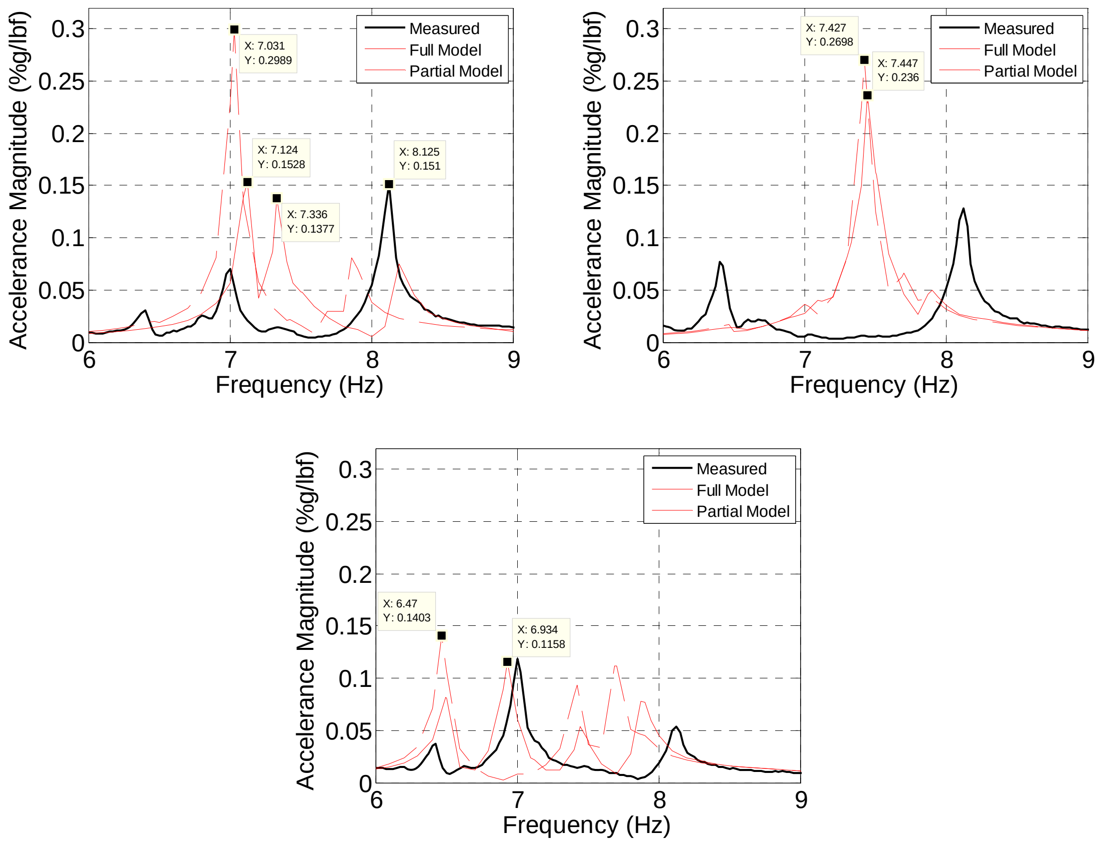


Figure 4.63: Accelerance Comparisons -- Full Model vs Partial Model. (a) Bay 1; (b) Bay 2; (c) Bay 3.

4.5.2 Response to Walking (Predictions Using Individual Footsteps)

Response to walking in each bay shown in Figure 4.55 was measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. Measured viscous modal damping was used in the model.

Walking in Bay 1. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.64(a) and (b). The step frequency for this measurement was 2.03 Hz (122 bpm) which was selected to cause resonance with Mode 6 (8.14 Hz) which was dominant in Bay 1 (Figure 4.61(a)). Tests were also conducted at the third subharmonic of the Mode 1 frequency (6.42 Hz) and the fourth subharmonic of the Mode 3 frequency (7.01 Hz), but these resulted in lower responses, although not by a wide margin. The measured acceleration spectrum indicates that the fourth harmonic of the walking force caused resonance with Mode 6. The third harmonic of the walking force also caused significant response between 6.0 Hz and 6.5 Hz. This explains the shape of the resonant build-up in Figure 4.64(a) which has the repeating pattern of two higher peaks followed by two smaller ones. Because two predicted accelerance peaks had approximately equal magnitude (Figure 4.61(a)), response history analyses were performed at the fourth subharmonic of both frequencies. The two predicted waveforms are shown in Figure 4.64(c) and (d). The maximum measured and predicted peak accelerations are 1.45%g and 1.34%g, respectively (Predicted / Measured = 0.924), indicating good agreement.

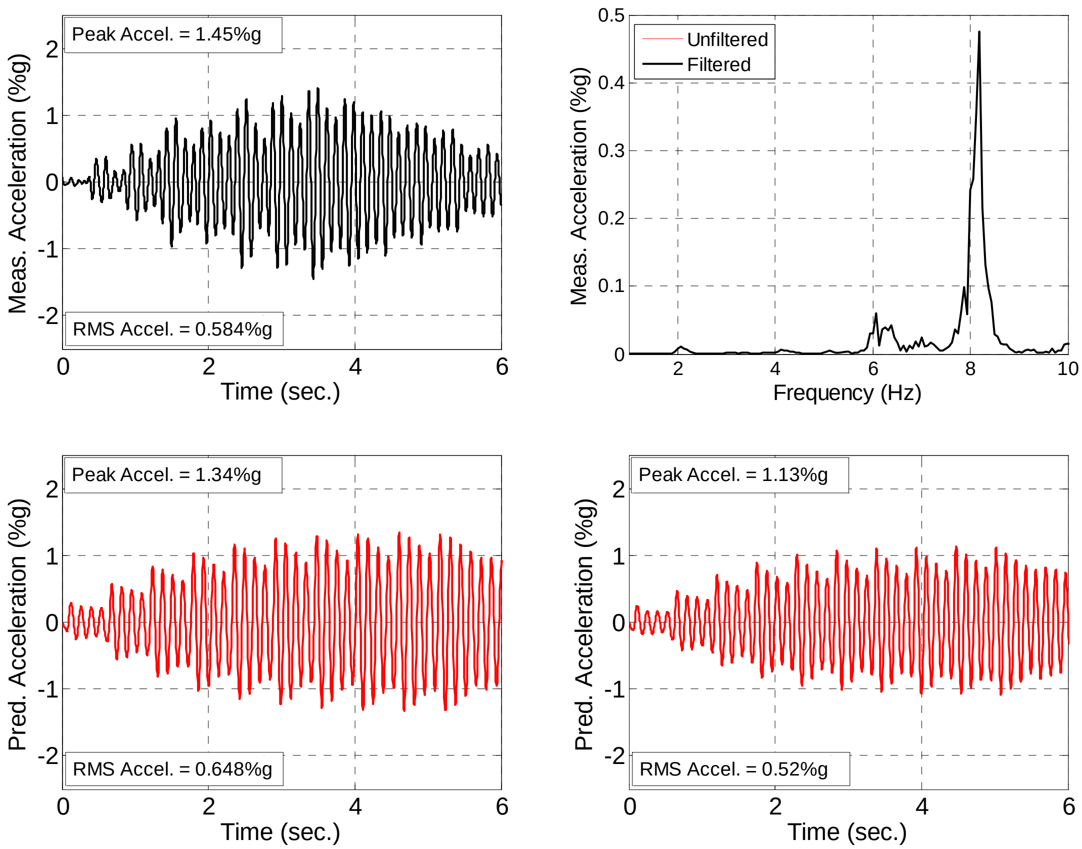


Figure 4.64: Measured and Predicted Responses to Walking in Bay 1, Riverside MOB. (a) Measured Waveform; (b) Measured Spectrum; (c) Predicted Waveform, Mode 3 Excited; (d) Predicted Waveform, Mode 4 Excited.

Walking in Bay 2. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.65(a) and (b). The step frequency for this measurement was 2.13 Hz (128 bpm) which was selected to cause resonance with Mode 1 (6.42 Hz) which was responsive in Bay 2 (Figure 4.61(b)). The measured acceleration spectrum indicates that the third and fourth harmonics of the walking force simultaneously caused resonance with Modes 1 and 6 (6.42 Hz and 8.14 Hz). As is the case for Bay 1 also, this explains the shape of the resonant build-up which has the repeating pattern of two higher peaks followed by two smaller ones. Tests were also conducted at the fourth subharmonic of the Mode 6 frequency (8.14 Hz), resulting in a peak acceleration slightly less than shown in Figure 4.65(a). Response history analysis was performed at the fourth subharmonic of the strongly dominant predicted Mode 5 frequency (7.45 Hz, mode shape shown in Figure 4.57(e)). The predicted waveform is shown in Figure 4.65(c). The maximum measured and predicted peak accelerations are 1.33%g and 2.31%g, respectively (Predicted / Measured = 1.74). The response was over-predicted because the fifth predicted mode over-estimates the displacement in Bay 2 compared to other areas of the structure, therefore under-estimating the effective mass. The ratio of predicted-to-measured peak acceleration approximately equals the ratio of predicted-to-measured peak accelerance magnitude shown in Figure 4.61(b). The behavior is also fundamentally different because the model did not predict two modes spaced to allow the third and fourth harmonics of the walking frequency to simultaneously cause resonance with both modes.

Walking in Bay 3. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.66(a) and (b). The step frequency for this measurement was 1.75 Hz (105 bpm) which was selected to cause resonance with Mode 3 (7.01 Hz) which was dominant in Bay 3 (Figure 4.61(c)). The measured acceleration spectrum indicates that the fourth harmonic of the walking force caused resonance with Mode 3. The waveform also indicates a fourth harmonic resonant build-up. Tests were also conducted at the third and fourth subharmonics of the Modes 1 and 6 frequencies (6.42 Hz and 8.14 Hz), respectively, resulting in peak accelerations slightly less than shown in Figure 4.65(a). Because four predicted modes (Modes 1, 2, 5, and 7) can provide significant response, as indicated by Figure 4.61(c), response history analyses were performed with walking at the appropriate subharmonic of each mode. The predicted waveforms are shown in Figure 4.66(c) through (f). The maximum measured and predicted peak accelerations are 1.03%g and 1.33%g, respectively (Predicted / Measured = 1.29), indicating reasonable agreement.

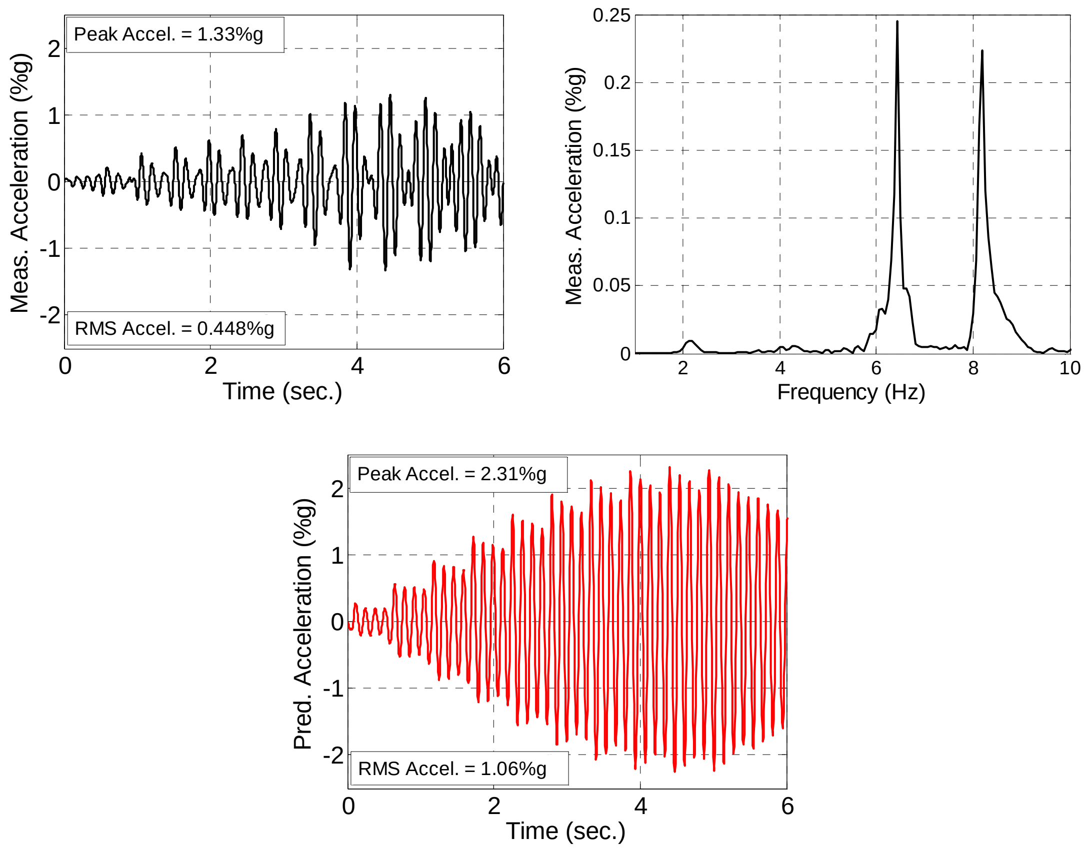


Figure 4.65: Measured and Predicted Responses to Walking in Bay 2, Riverside MOB. (a) Measured Waveform; (b) Measured Spectrum; (c) Predicted Waveform, Mode 5 Excited.

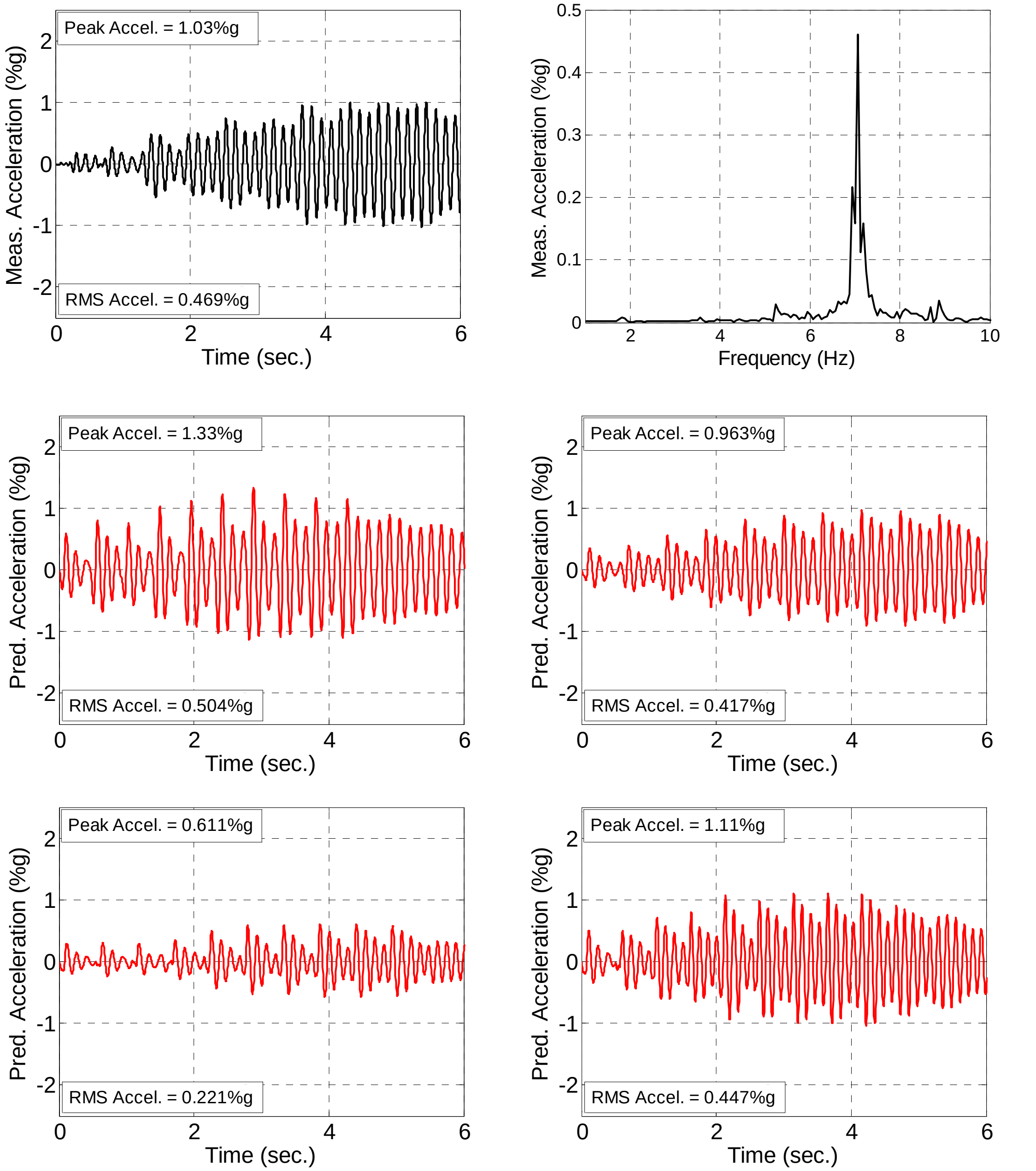


Figure 4.66: Measured and Predicted Responses to Walking in Bay 3, Riverside MOB. (a) Measured Waveform; (b) Measured Spectrum; (c) Predicted Waveform, Mode 1 Excited; (d) Predicted Waveform, Mode 2 Excited; (e) Predicted Waveform, Mode 5 Excited; (f) Predicted Waveform, Mode 7 Excited.

Walking in Bay 4. The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.67(a) and (b). The step frequency for this measurement was 2.21 Hz (133 bpm) which was selected to cause resonance with Mode 2 (6.61 Hz) which was responsive in Bay 4 (Figure 4.61(d)). The measured acceleration spectrum indicates that the third harmonic of the walking force excited Mode 2 and, to a lesser extent, several other higher frequency modes. The measured waveform indicates no resonant build-up. Tests were also conducted at the fourth subharmonic of the following frequencies: 6.61 Hz, 7.14 Hz, 7.46 Hz, and 8.14 Hz, all resulting in lower accelerations than the one shown in Figure 4.67(a). Response history analysis was performed at the fourth subharmonic of the strongly dominant predicted Mode 7 frequency (7.88 Hz, shape shown in Figure 4.57(g)). The predicted waveform is shown in Figure 4.67(c). The maximum measured and predicted peak accelerations are 0.775%g and 2.23%g, respectively (Predicted / Measured = 2.88). The response was over-predicted because the seventh predicted mode over-estimates the displacement in Bay 4 compared to other areas of the structure, therefore under-estimating the effective mass. The ratio of predicted-to-measured peak acceleration approximately equals the ratio of predicted-to-measured peak accelerance magnitude shown in Figure 4.61(d). The behavior is also fundamentally different. The measured response is almost completely lacking a resonant build-up, most likely due to the responses of several modes between 7 Hz and 10 Hz, which prevent resonant build-up of Mode 2. The model did not predict this interaction of modes, so the resonant build-up was over-predicted.

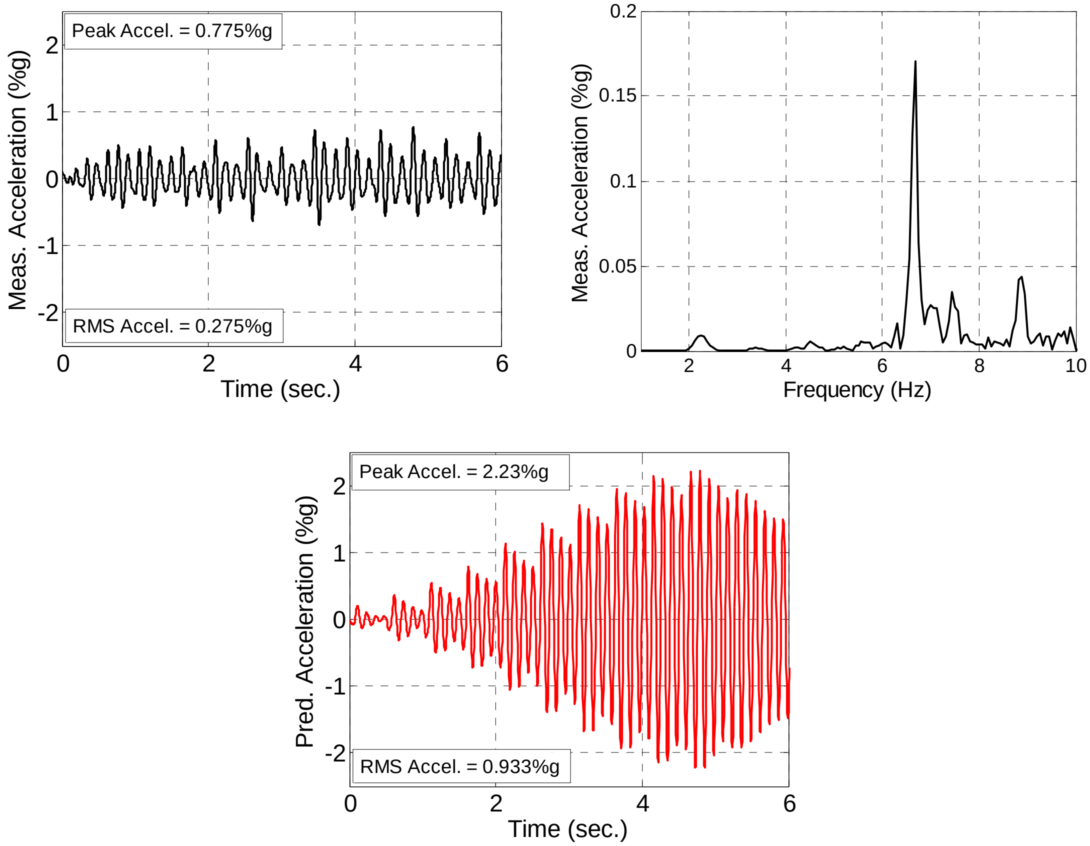


Figure 4.67: Measured and Predicted Responses to Walking in Bay 4, Riverside MOB. (a) Measured Waveform; (b) Measured Spectrum; (c) Predicted Waveform, Mode 7 Excited.

4.5.3 Response to Walking (Predictions Using Fourier Series Loading)

Response to walking in each bay shown in Figure 4.55 was measured using the methods described in Chapter 2 and predicted using the Fourier series loading method described in Chapter 3. Measured viscous modal damping was used in the model. Responses were predicted using all four Fourier series terms and with one term—the one that matches the natural frequency.

Walking in Bay 1. The measured walking acceleration waveform for the test with maximum response is shown in Figure 4.68(a). The response was predicted using two separate response history analyses with the fourth harmonic of the walking force exciting the predicted Modes 3 and 4. The predicted waveforms (third mode gave a slightly higher response), using four terms and one term, respectively, are shown in Figure 4.68(b) and (c). The response predicted using only one Fourier series term was not appreciably different from the response predicted using all four terms. The predicted-to-measured peak acceleration is 0.95 using all four terms and 0.90 using only one term. The ratios indicate good agreement, but the behaviors shown in the waveforms are fundamentally different, however, due to the interaction of predicted modes.

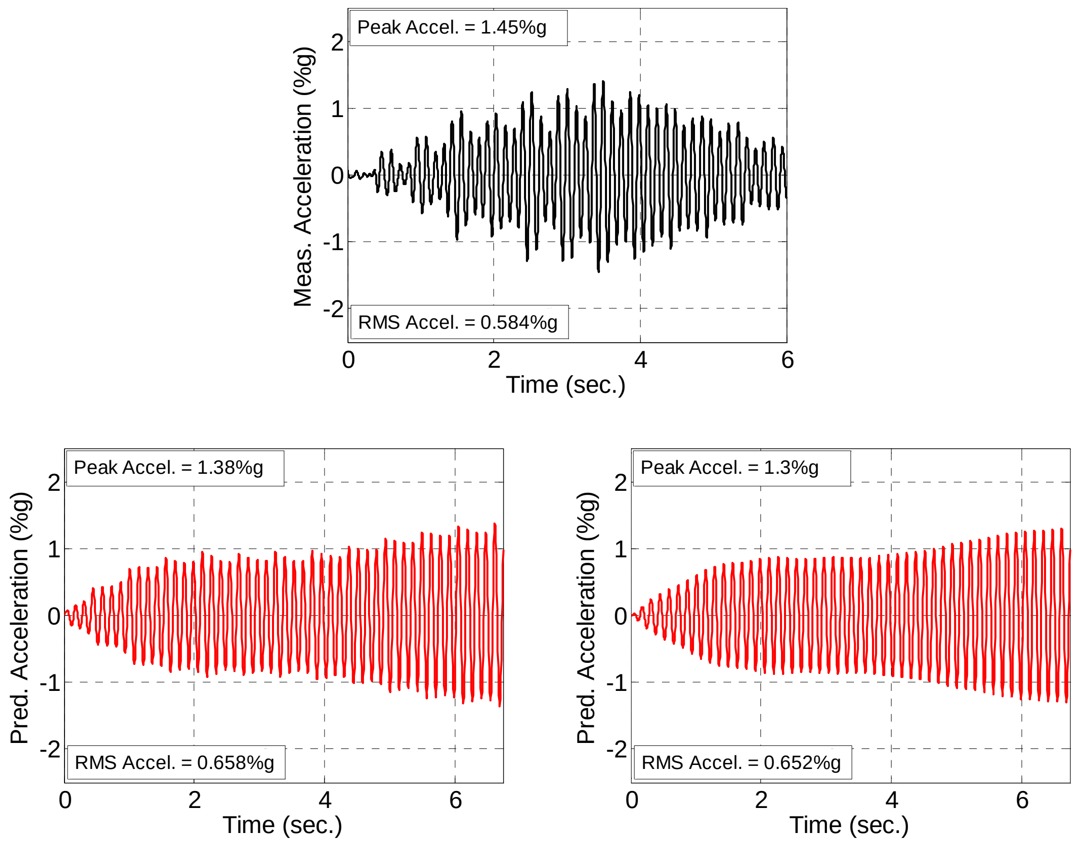


Figure 4.68: Acceleration Response to Walking in Bay 1, Mode 3 Excited, Fourier Series Loading, Riverside MOB. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform (4 terms); (c) Predicted Acceleration Waveform (1 term).

Walking in Bay 2. The measured walking acceleration waveform for the test with maximum response is shown in Figure 4.69(a). The response was predicted with the fourth harmonic of the walking force exciting the predicted Mode 5. The predicted waveforms, using four terms and one term, respectively, are shown in Figure 4.69(b) and (c). The response predicted using only one Fourier series term was not appreciably different from the response predicted using all four terms. The predicted-to-measured peak acceleration is 1.71 using all four terms and 1.67 using only one term. The model significantly over-predicted the acceleration because the fifth predicted mode had an accelerance peak that was far higher than measured. The measured and predicted behaviors are also very different. The measured behavior indicates a resonant build-up with both the third and fourth harmonics of the walking force whereas the predicted behavior was a simple fourth harmonic resonant build-up.

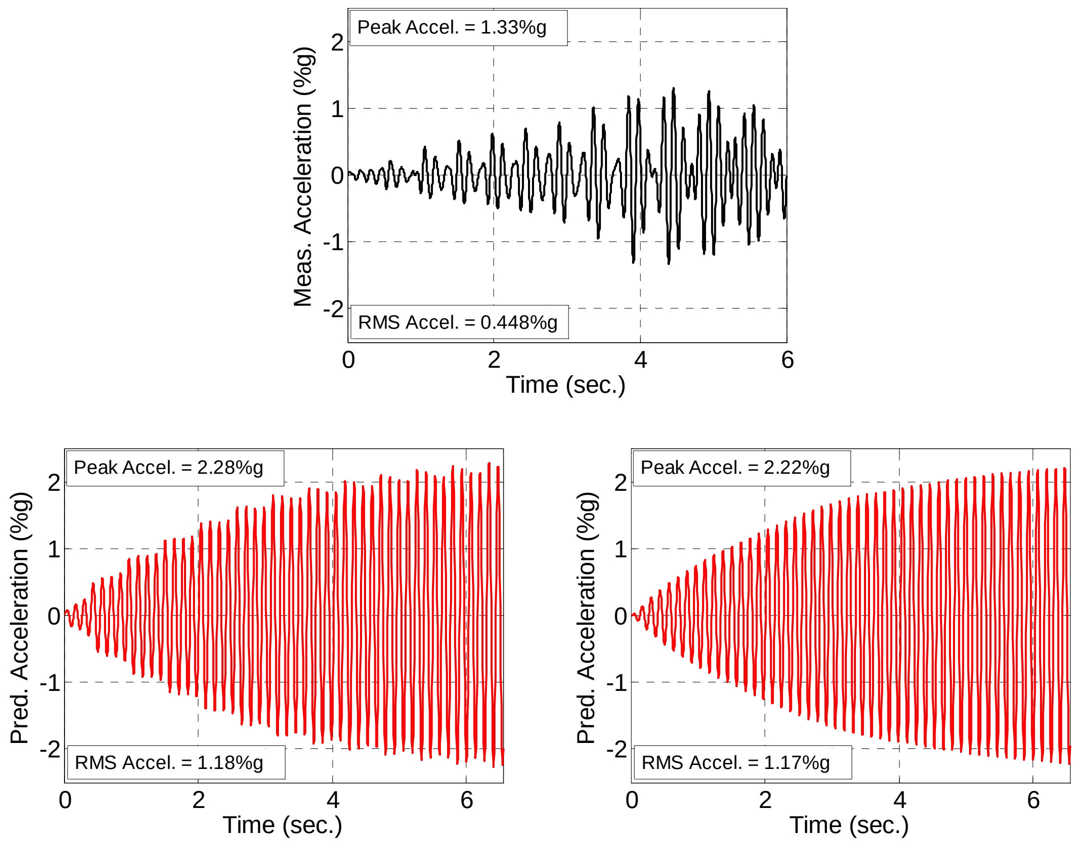


Figure 4.69: Acceleration Response to Walking in Bay 2, Fourier Series Loading, Riverside MOB. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform (4 terms); (c) Predicted Acceleration Waveform (1 term).

Walking in Bay 3. The measured walking acceleration waveform for the test with maximum response is shown in Figure 4.70(a). The response was predicted with the third harmonic of the walking force exciting the predicted Mode 1. Analyses using harmonic loads matching Modes 2, 5, and 7 (the other modes with significant response in Bay 3) predicted lower accelerations. The predicted waveforms, using four terms and one term, respectively, are shown in Figure 4.70(b) and (c). In this case, the response predicted using only one Fourier series term was appreciably different from the response predicted using all four terms. The predicted-to-measured peak acceleration ratio is 1.17 using all four terms and 0.918 using only one term, indicating good agreement in both cases. The fundamental behavior is different partially because the measured waveform was taken from a test that excited the fourth subharmonic of a natural frequency whereas the prediction was developed using the third subharmonic.

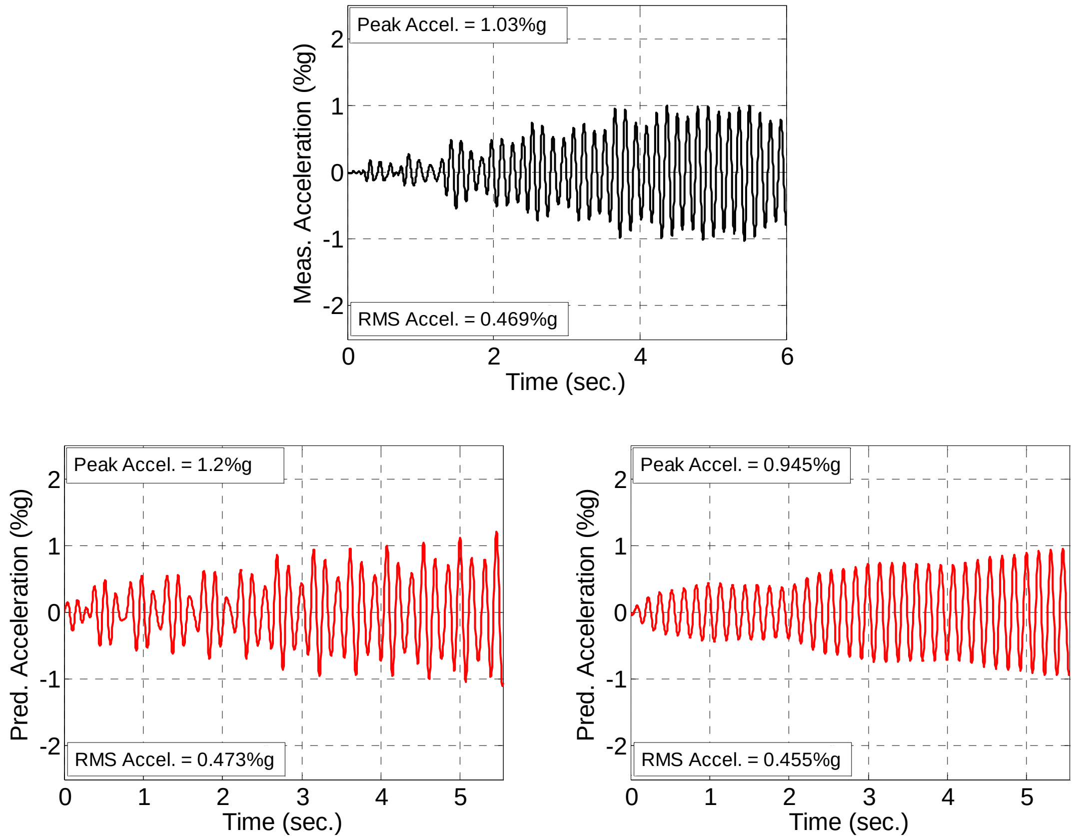


Figure 4.70: Acceleration Response to Walking in Bay 3, Fourier Series Loading, Riverside MOB. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform (4 terms); (c) Predicted Acceleration Waveform (1 term).

Walking in Bay 4. The measured walking acceleration waveform for the test with maximum response is shown in Figure 4.71(a). The response was predicted with the fourth harmonic of the walking force exciting the predicted Mode 7. An analysis using the third harmonic load matching Mode 1 (the other mode with significant predicted response in Bay 4) predicted a lower acceleration. The predicted waveforms, using four terms and one term, respectively, are shown in Figure 4.71(b) and (c). The response predicted using only one Fourier series term was only slightly different from the response predicted using all four terms. The predicted-to-measured peak acceleration ratio is 2.66 using all four terms and 2.52 using only one term, indicating significant over-estimation in both cases. The responses were significantly over-predicted because the accelerance peak magnitude was significantly over-predicted (Figure 4.61).

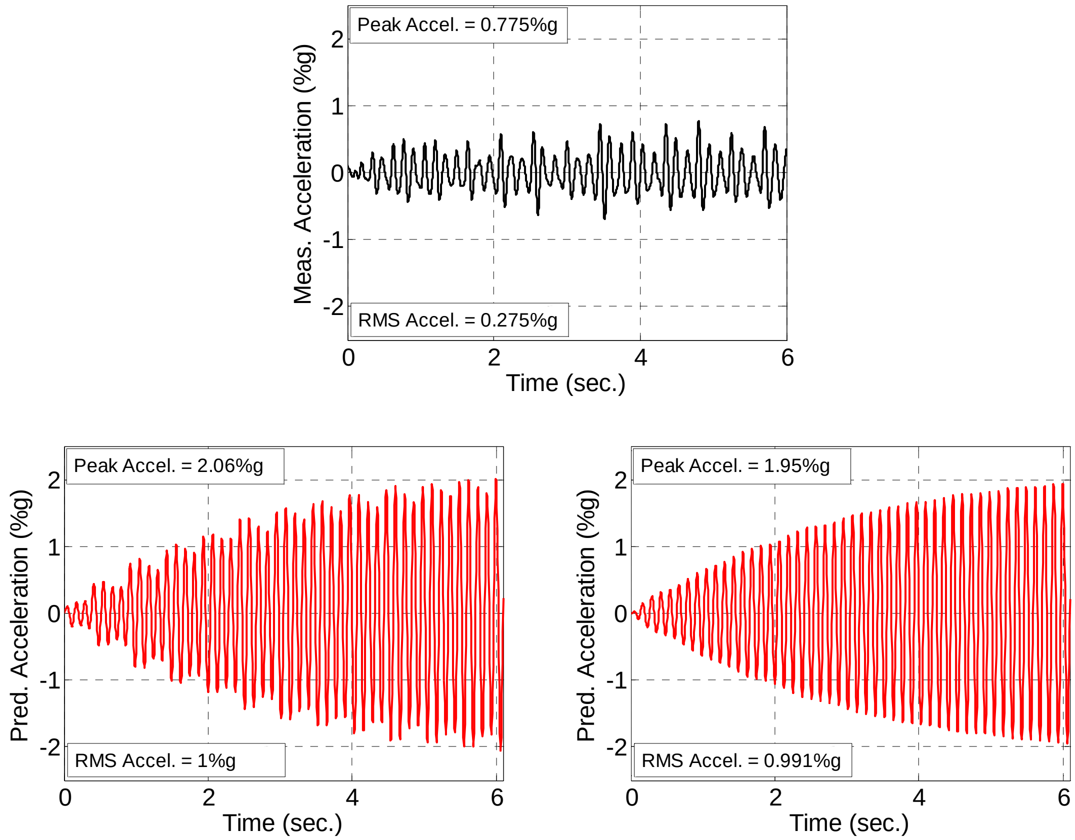


Figure 4.71: Acceleration Response to Walking in Bay 4, Fourier Series Loading, Riverside MOB. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform (4 terms); (c) Predicted Acceleration Waveform (1 term).

4.5.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. The response was predicted using the predicted FRF magnitude and the measured FRF magnitude. The predicted and measured mode shapes, and therefore accelerance peak magnitudes did not correspond in most bays. Therefore, the dominant predicted and dominant measured accelerance peak magnitudes were used in each bay to predict the acceleration due to walking. In bays with two similarly responsive modes, both modes were used to predict the acceleration, with the maximum acceleration due to walking reported here.

In Bay 1, using the predicted FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.153 %g/lbf at a frequency of 7.12 Hz. The walking path was 30 ft and the damping ratio was 0.54% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 1.38%g whereas the measured peak acceleration due to walking was 1.45%g. The ratio of predicted-to-measured accelerations is 0.952, indicating a rare (for this research) under-prediction of acceleration due to walking.

In Bay 1, using the measured FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.151 %g/lbf at a frequency of 8.125 Hz. The predicted peak acceleration due to walking was 1.52 %g. The ratio of predicted-to-measured accelerations is 1.05, indicating a very accurate prediction.

In Bay 2, using the predicted FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.236 %g/lbf at a frequency of 7.45 Hz. The walking path was 30 ft and the damping ratio was 0.54% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 2.21%g whereas the measured peak acceleration due to walking was 1.33 %g. The ratio of predicted-to-measured accelerations is 1.66.

In Bay 2, using the measured FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.128 %g/lbf at a frequency of 8.125 Hz. The predicted peak acceleration due to walking was 1.28 %g. The ratio of predicted-to-measured accelerations is 0.962. Bay 2 is another excellent example of predicted accelerance peak magnitude over-predictions directly leading to a significant over-prediction of acceleration due to walking. When the measured FRF was used, the predicted acceleration almost matched the measured one.

In Bay 3, using the predicted FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.116 %g/lbf at a frequency of 6.93 Hz. The walking path was 30 ft and the damping ratio was 0.54% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 1.02%g whereas the measured peak acceleration due to walking was 1.03 %g. The ratio of predicted-to-measured accelerations is 0.99.

In Bay 3, using the measured FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.118 %g/lbf at a frequency of 7.0 Hz. The predicted peak acceleration due to walking was 1.05 %g. The ratio of predicted-to-measured accelerations is 1.02. The response was very accurately predicted using the predicted and measured FRF magnitudes.

In Bay 4, using the predicted FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.192 %g/lbf at a frequency of 7.88 Hz. The walking path was 30 ft and the damping ratio was 0.54% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 1.88%g whereas the measured peak acceleration due to walking was 0.775 %g. The ratio of predicted-to-measured accelerations is 2.43.

In Bay 4, using the measured FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.0770 %g/lbf at a frequency of 7.43 Hz. The predicted peak acceleration due to walking was 0.718 %g. The ratio of predicted-to-measured accelerations is 0.927. Like Bay 2, Bay 4 is an excellent example of accelerance peak magnitude over-prediction causing over-prediction of acceleration due to walking.

4.6 First Bank and Trust Building

One floor of a second building under construction was vibration tested in October 2006. The building is four stories, 16,000 sq. ft, and is shown in its finished condition in Figure 4.72. Figure 4.73 shows its condition at the time of vibration tests.

The top floor was chosen because steel stud partition walls were partially constructed at the other levels and because its topside supported less construction material than the other floors. The slab topside was mostly clear of construction material and the underside supported only very minimal piping and ductwork. See Figure 4.74 and Figure 4.75. Full height steel studs were installed above and below the slab between B-3 and D-4, shown in the Figure 4.76. Otherwise, no steel stud walls were installed except at the exterior walls.

The floor was built using a thin non-composite slab supported by open-web steel joists and hot-rolled steel beams and girders. The normal weight concrete slab is 3 in. total thickness on 9/16 in. form deck. Bay sizes are very irregular. The bay that was the subject of most of the tests (indicated in Figure 4.76) was approximately 22 ft by 32 ft.

Figure 4.72: First Bank & Trust Building (Finished)

Figure 4.73: First Bank & Trust Building (During Vibration Tests)

Figure 4.74: First Bank & Trust Building Slab Topside Condition

Figure 4.75: First Bank & Trust Building Slab Underside

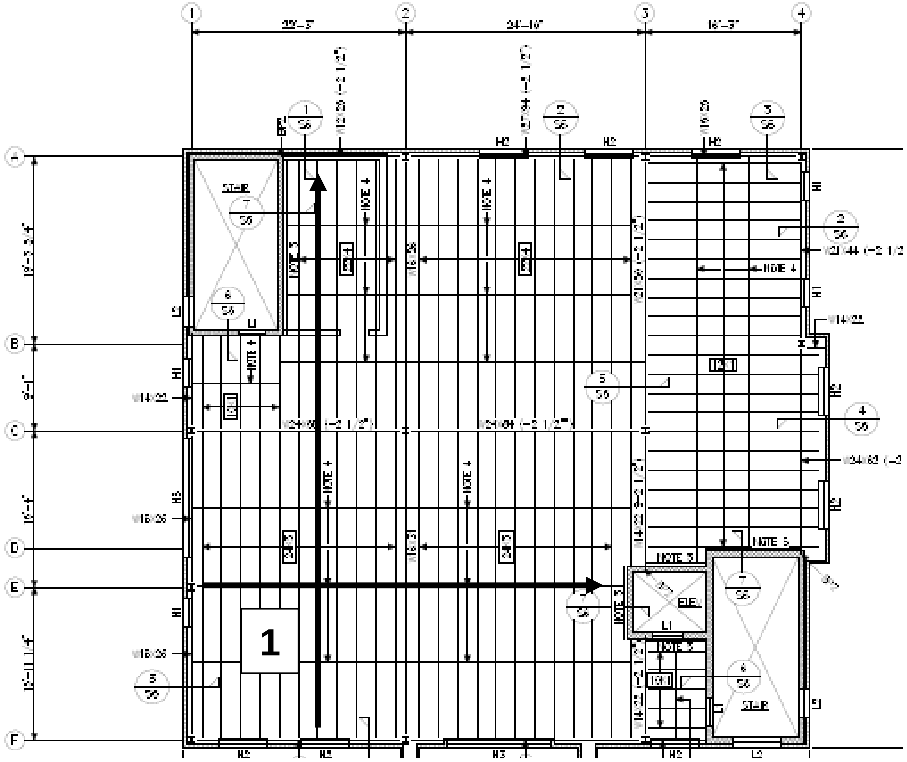


Figure 4.76: First Bank & Trust Floor Plan

4.6.1 Modal Properties

Modal properties were measured using the methods described in Chapter 2 and predicted using the methods described in Chapter 3. The shaker was placed at the center of C-1/F-2, A-2/C-3, and C-2/F-3. Accelerations were measured at the centers of bays, at midspan of interior and spandrel beams and girders. In the model, the spandrel was modeled using the recommendation by Barrett (2006): multiply the transformed moment of inertia by 2.5.

The measurements and predictions both indicate that the floor has several vibration modes that can be excited by walking. The first five measured natural frequencies are 8.32 Hz, 8.56 Hz, 8.92 Hz, 9.55 Hz, and 10.6 Hz and first three predicted natural frequencies are 8.18 Hz, 8.81 Hz, and 9.51 Hz. Predicted modes above 9.51 Hz are double-curvature within bays and do not correspond to measured modes, so are not shown. The measured and predicted mode shapes are shown in Figure 4.78 and Figure 4.79. Interestingly, two measured vibration modes have almost the same shape: Modes 1 and 4 (8.32 Hz and 9.55 Hz), respectively. The difference between these two modes is the direction of the slab movement at areas of little response. In Mode 1, these areas are moving in the same direction as the areas with greater response whereas in Mode 4, these areas are moving in the opposite direction. Measured Mode 3 (8.92 Hz) roughly corresponded to predicted Mode 3 (9.51 Hz) and measured Mode 5 (10.6 Hz) corresponded to predicted Mode 2 (8.81 Hz). Otherwise, the predicted and measured modes were of different shapes. It is also interesting that the model did not predict a mode similar to the measured Mode 2 (one bay up, other bay down) because it did predict a mode similar to the measured Mode 1 and 4 (both bays up).

Measured mode shape starburst plots are shown in Figure 4.80. Modes 3 through 5 are quasi-real, indicating that damping is spatially distributed in a similar manner as stiffness and mass. Modes 1 and 2 are still only slightly complex.

Measured and predicted driving point accelerance FRF magnitudes were also compared. Measured damping was used in the model to allow a valid comparison. EMA FRF curve-fitting was used to determine the viscous modal damping ratios shown in

Table 4.13: Damping Ratios, First Bank & Trust Building. The curve-fit FRFs for several DOF in the three tested bays are shown in Figure 4.81, indicating a very precise match of the parametric model and estimated FRFs. The damping ratios are significantly higher than measured for the other specimens. It is speculated that damping is higher because the joist bottom chord extensions connect to beams at the spandrel, requiring movement of the cladding during floor vibration. See Figure 4.77, which shows the bottom chord extensions and cladding connections to the underside of one of the spandrel girders. Because the floor only consists of two bays in the north-south direction, most bays have one girder connected to the cladding in this manner. Floor vibration requires vertical and horizontal movement of the girder bottom flange at these locations. Also, the measured Mode 5 had a significantly lower damping ratio and was less complex than the other modes and it is primarily associated with movement in Bay C-2/F-3. The plan-south girder in this bay is not connected to cladding over much of its length because it is located at a large wall opening.

Constant hysteretic damping with mass proportional coefficient set equal to zero was used for the FRF predictions in the three bays. Stiffness proportional coefficients were 0.0265, which is double the average of the first two measured modal damping ratios. This was chosen because the measured and predicted modes did not correspond in most cases, as discussed in a previous paragraph. The comparisons are shown in Figure 4.82 and summarized in Table 4.14, indicating that the model very significantly over-predicted the accelerance peak magnitude in two out of the three tested bays. The reason for this is shown in Figure 4.83, which shows the predicted mode shapes. These three mode shapes have significant movement only in one bay whereas most of the measured mode shapes had significant movement in at least two bays. In Bay C-1/F-2, there is no vibration mode that has the vast majority of the motion only in that bay. In bay C-2/F-3, such a mode exists in reality (Mode 5), but it has a much higher frequency than predicted (10.6 Hz vs 8.81 Hz).

Table 4.13: Damping Ratios, First Bank & Trust Building

Description	Damping (% of Critical)
Mode 1	1.15
Mode 2	1.50
Mode 3	1.40
Mode 4	1.41
Mode 5	0.95

Table 4.14: Measured and Predicted Accelerance Peak Magnitudes, First Bank & Trust Building

Bay	Accelerance Peak Magnitude		
	Measured (%g/lbf)	Predicted (%g/lbf)	Pred. / Meas.
C-1/F-2	0.181	0.454	2.51
C-2/F-3	0.261	0.407	1.56
A-2/F-3	0.162	0.460	2.84

Figure 4.77: First Bank & Trust Building Spandrel Condition

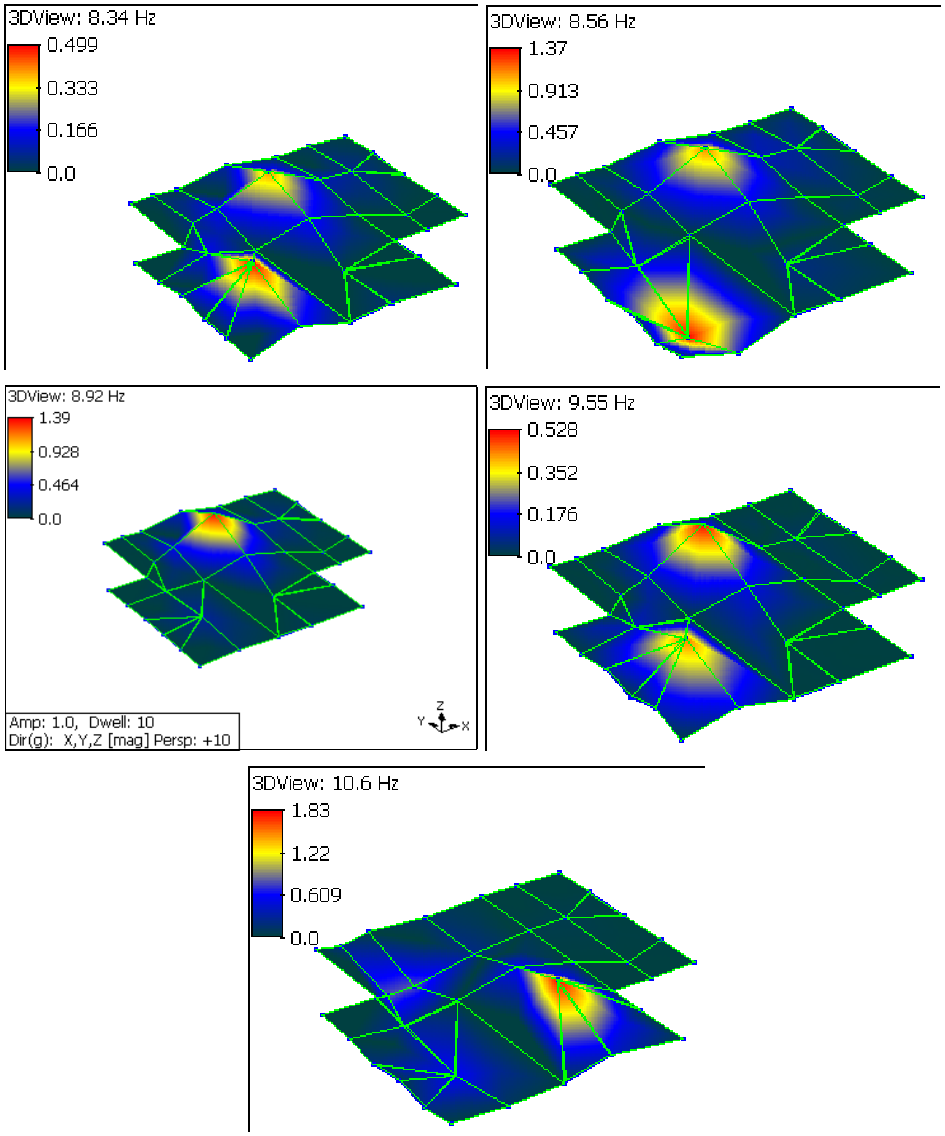


Figure 4.78: First Bank & Trust Building Measured Vibration Mode Shapes. (a) Mode 1: 8.32 Hz; (b) Mode 2: 8.56 Hz; (c) Mode 3: 8.92 Hz; (d) Mode 4: 9.55 Hz; (e) Mode 5: 10.6 Hz.

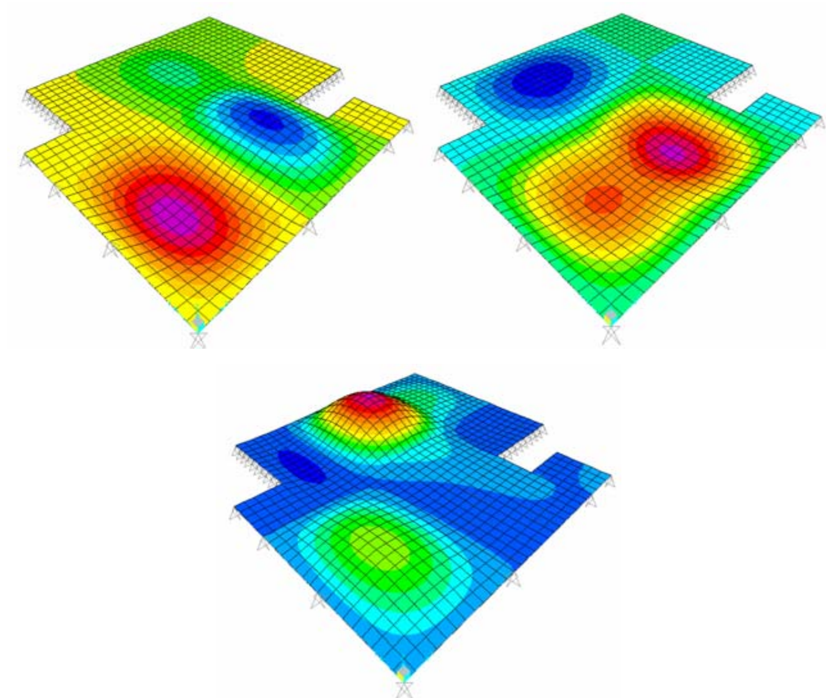


Figure 4.79: First Bank & Trust Building Predicted Mode Shapes. (a) Mode 1: 8.18 Hz; (b) Mode 2: 8.81 Hz; (c) Mode 3: 9.55 Hz.

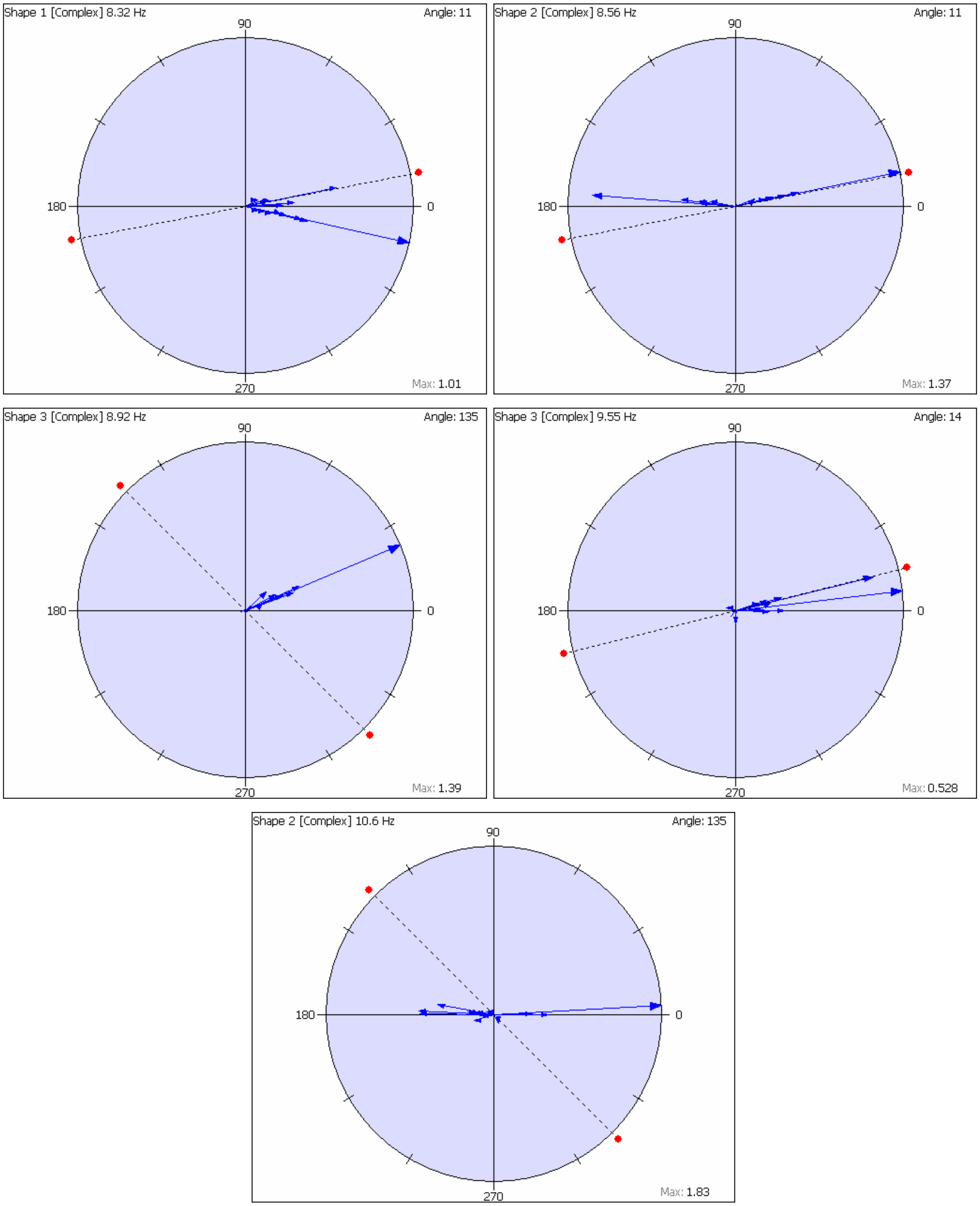


Figure 4.80: First Bank & Trust Building Starburst Plots

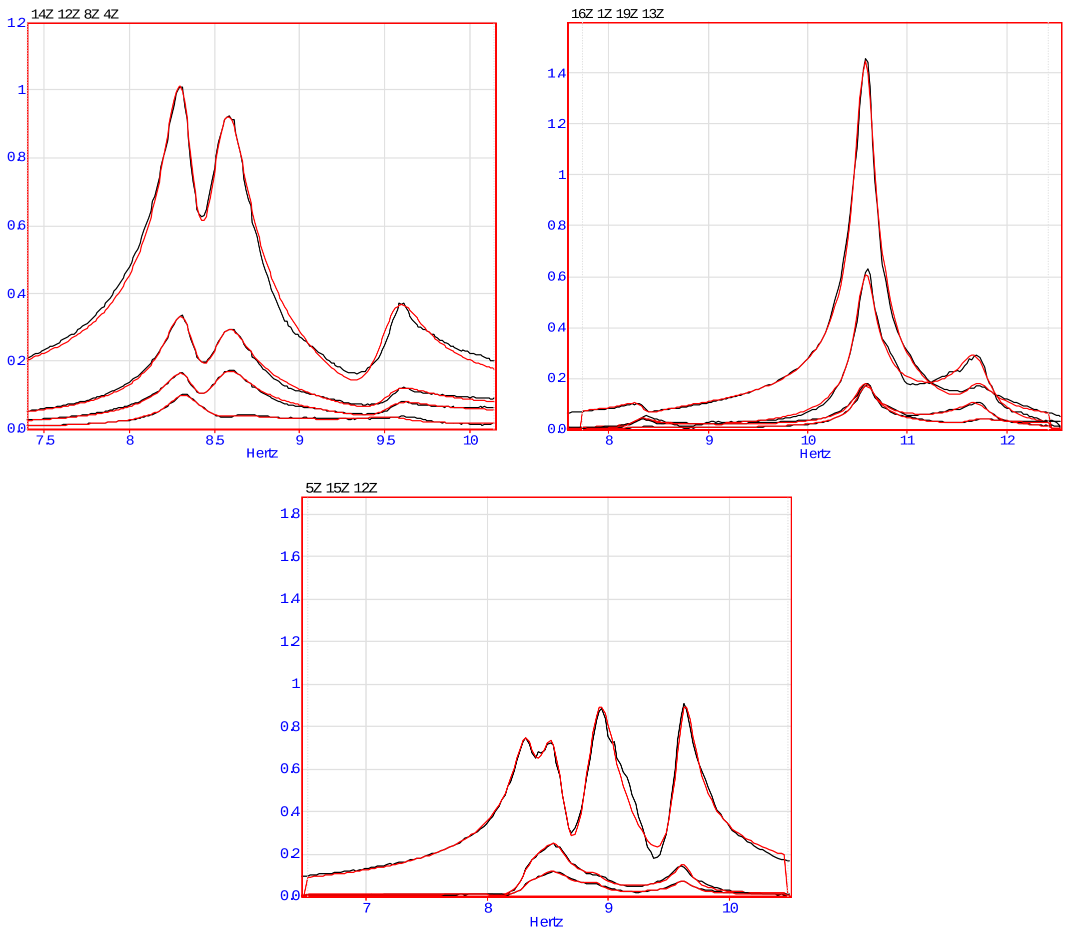


Figure 4.81: Curve-Fit FRFs for the First Bank & Trust. (a) Bay 1 (C-1/F-2); (b) C-2/F-3; (c) A-2/C-3.

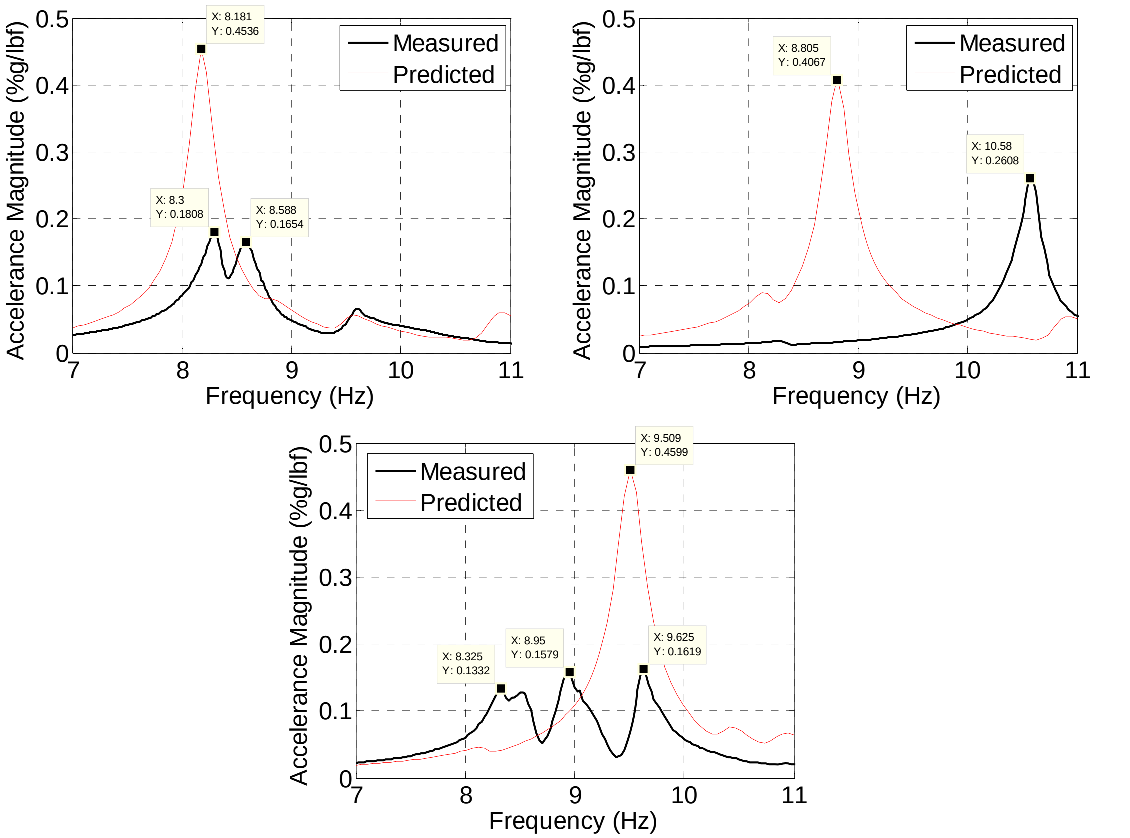


Figure 4.82: Measured and Predicted Accelerance FRF Magnitudes, First Bank & Trust Building (a) Shaker at C-1/F-2; (b) Shaker at C-2/F-3; (c) Shaker at A-2/C-3.

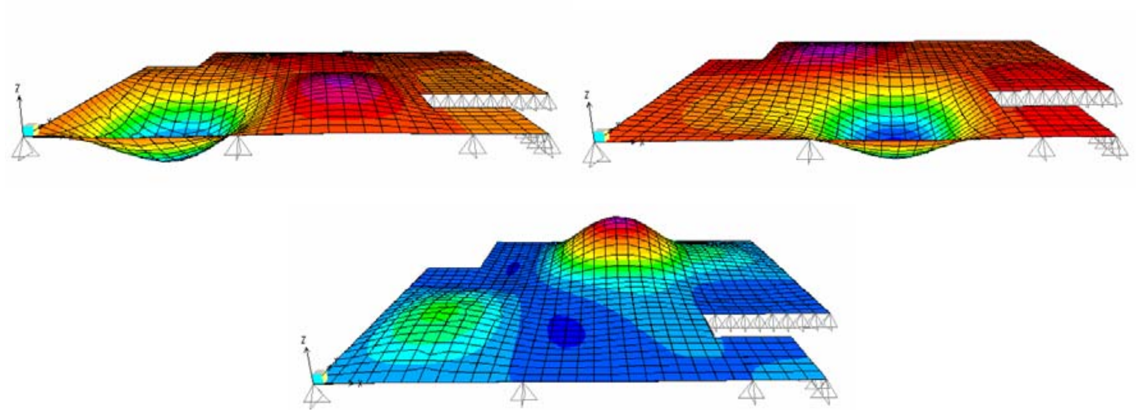


Figure 4.83: First Bank & Trust Predicted Mode Shapes (Alternate View). (a) Mode 1; (b) Mode 2; (c) Mode 3.

4.6.2 Response to Walking (Predictions Using Individual Footsteps)

Response to walking in bay C-1/F-2, shown in Figure 4.76, was measured using the methods described in Chapter 2 and predicted using the individual footstep application method described in Chapter 3. Measured viscous modal damping was used in the model.

The measured walking acceleration waveform and spectrum for the test with maximum response are shown in Figure 4.84 (a) and (b). The step frequency for this measurement was 2.11 Hz (127 bpm). Tests were conducted at numerous frequencies between (2.01 Hz) 121 bpm and (2.18Hz) 131 bpm, but these resulted in lower responses. The measured waveform indicates a fourth harmonic resonant build-up. The measured acceleration spectrum indicates that the fourth harmonic of the walking force caused resonance with the measured Mode 2 (8.56 Hz) and contains a smaller contribution from the Mode 1 (8.32 Hz). Response history analysis was performed at the fourth subharmonic of the dominant predicted Mode 1 (8.12 Hz). The predicted waveform is shown in Figure 4.84(c). The maximum measured and predicted peak accelerations are 2.44%g and 4.95%g, respectively (Predicted / Measured = 2.03), indicating a significant over-prediction. The reason for the over-prediction is the very significantly under-predicted effective mass as indicated by the over-predicted accelerance FRF magnitude shown in Figure 4.82(a). It is also obvious from looking at the measured and predicted waveforms that the actual walker was not able to maintain the resonant build-up as long as the electronic walker did.

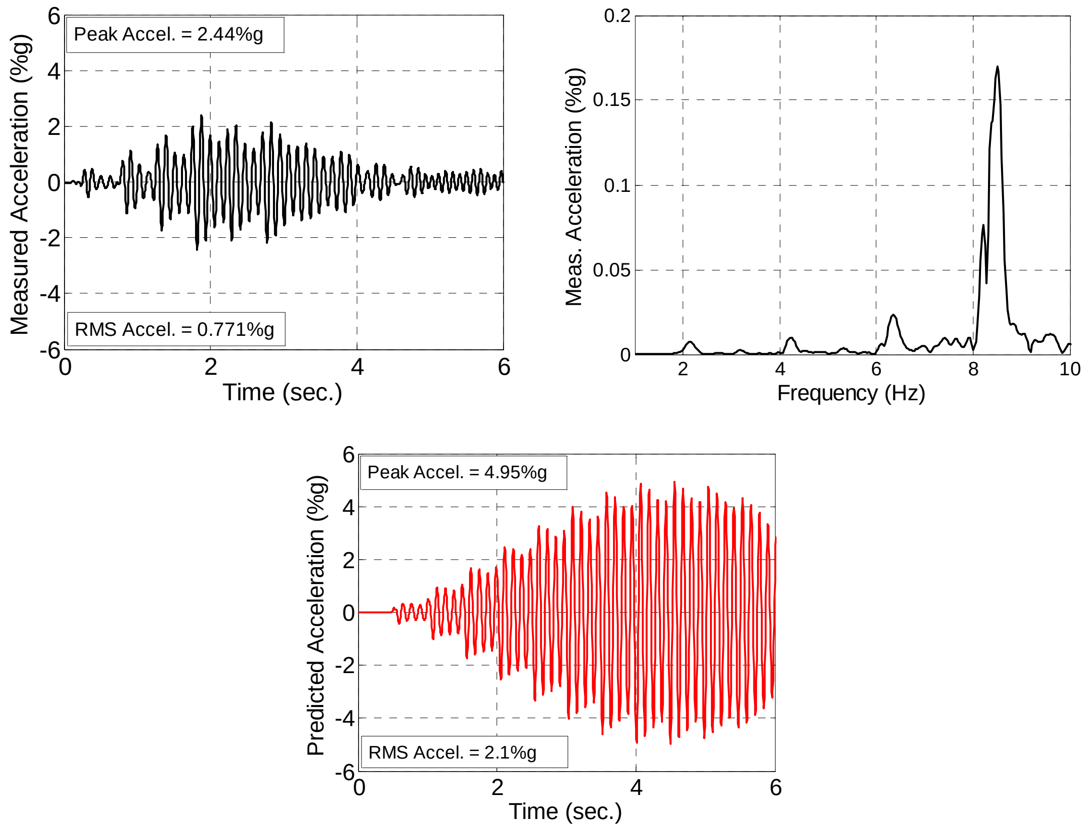


Figure 4.84: Measured and Predicted Responses to Walking in Bay C-1/F-2, First Bank & Trust Building. (a) Measured Waveform; (b) Measured Spectrum; (c) Predicted Waveform, Predicted Mode 1 Excited;

4.6.3 Response to Walking (Predictions Using Fourier Series Loading)

Response to walking in Bay C-1/F-2 was measured using the methods described in Chapter 2 and predicted using the Fourier series loading method described in Chapter 3. Measured viscous modal damping was used in the model. Responses were predicted using all four Fourier series terms and with one term—the one that matches the natural frequency.

The measured walking acceleration waveform for the test with maximum response is shown in Figure 4.85(a). The response was predicted using two separate response history analyses with the fourth harmonic of the walking force exciting the predicted Mode 1. The predicted waveforms, using four terms and one term, respectively, are shown in Figure 4.85(b) and (c). The response predicted using only one Fourier series term was not appreciably different from the response predicted using all four terms. The predicted-to-measured peak acceleration is 1.75 using all four terms and

1.66 using only one term. The ratios indicate significant over-prediction, undoubtedly due to the accelerance FRF over-prediction.

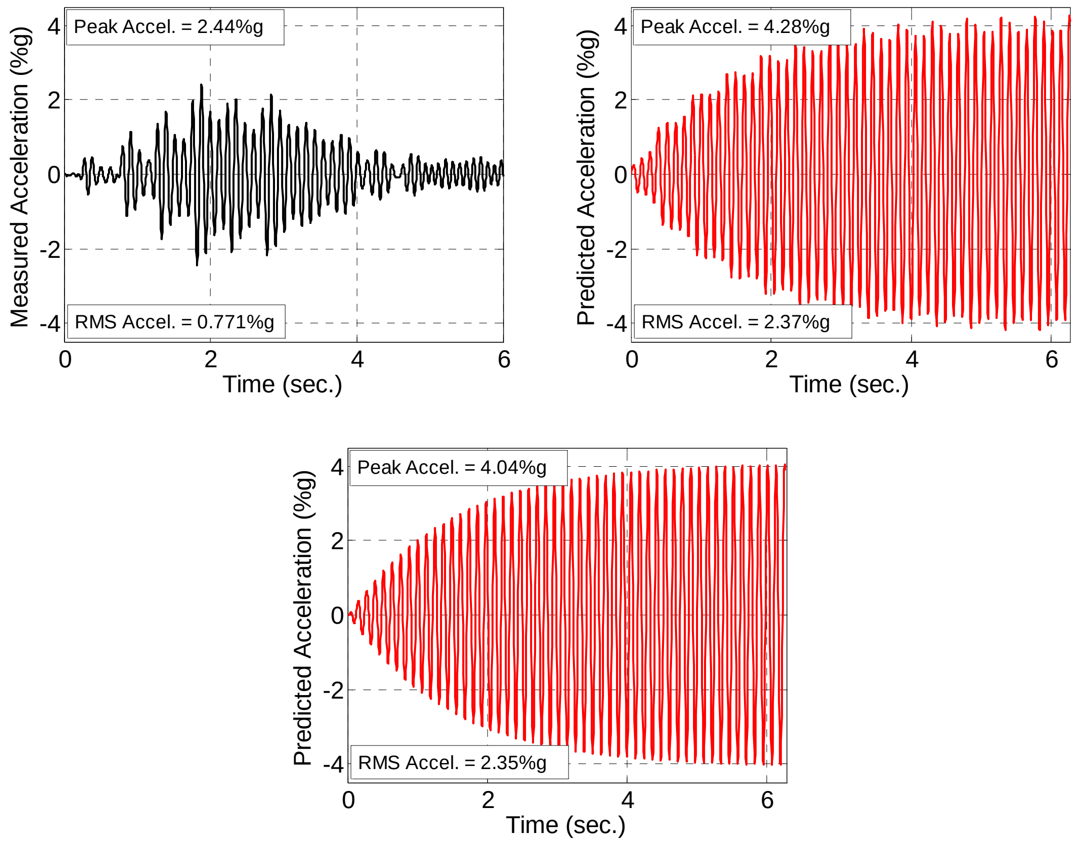


Figure 4.85: Acceleration Response to Walking in Bay C-1/F-2, Mode 1 Excited, Fourier Series Loading, First Bank & Trust Building. (a) Measured Acceleration Waveform; (b) Predicted Acceleration Waveform (4 terms); (c) Predicted Acceleration Waveform (1 term).

4.6.4 Response to Walking (Predictions Using Simplified Frequency Domain Procedure)

Response to walking was also predicted using the Simplified Frequency Domain Procedure described in Chapter 3. The response was predicted using the predicted FRF magnitude and the measured FRF magnitude. The predicted and measured mode shapes, and therefore acceleration peak magnitudes did not correspond in the tested bay. Two modes were similarly responsive, so both modes were used to predict the acceleration due to walking, with the maximum acceleration due to walking reported here.

Using the predicted FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.454 %g/lbf at a frequency of 8.18 Hz. The walking path was 30 ft and the damping ratio was 1.33% of critical. Using the method described in Section 3.5, the acceleration is predicted to be 4.11%g whereas the measured peak acceleration due to walking was 2.44%g. The ratio of predicted-to-measured accelerations is 1.68.

Using the measured FRF, the maximum predicted acceleration was due to the accelerance peak magnitude of 0.181%g/lbf at a frequency of 8.3 Hz. The predicted peak acceleration due to walking was 1.66%g. The ratio of predicted-to-measured accelerations is 0.68, which was the only significant under-prediction of acceleration response in this research.

4.7 Summary of Comparisons

This section contains summaries of the comparisons of measured and predicted natural frequencies, accelerance peak magnitudes, and acceleration response to walking for all specimens which are included in this research.

Table 4.15 summarizes the measured and predicted natural frequencies. The average ratio of predicted-to-measured natural frequency is 0.992. Excluding First Bank & Trust Building Mode 5, the predictions are all within 10% of the corresponding measured values. The coefficient of variation (COV) equals 7.05%, indicating only slight data dispersion. Several measured and predicted modes for the Riverside MOB and First Bank & Trust Building did not have corresponding shapes, so could not be compared. However, for both of these buildings, the first few measured and predicted natural frequencies were in the same general ranges. Overall, the models provided excellent natural frequency predictions.

Table 4.16 summarizes the measured and predicted driving point accelerance peak magnitudes which were computed using measured damping ratios. The average ratio of predicted-to-measured accelerance peak magnitude is 1.49. The model significantly over-predicted the peak magnitude in approximately half of the cases and moderately under-predicted the peak magnitude in only one case. In most cases of over-prediction, the models incorrectly predicted that the bay containing the driving point displaced much more than all surrounding bays, resulting in an under-prediction of effective mass. The COV equals 43.9%, indicating significant dispersion.

Table 4.15: Summary of Natural Frequency Comparisons

Specimen	Description	Natural Frequency Measured (Hz)	Natural Frequency Predicted (Hz)	Natural Frequency Predicted / Measured
Long Span Composite Slab Specimen	Mode 1	4.98	5.20	1.04
Long Span Composite Slab Mockup	Bay 1, Mode 1	5.55	5.56	1.00
	Bay 1, Mode 2	5.75	6.00	1.04
	Bay 2, Mode 1	6.00	5.72	0.953
	Bay 2, Mode 2	6.64	6.56	0.988
Square-End Joist Footbridge	Mode 1	7.75	7.69	0.992
Shear-Connected Joist Footbridge	Mode 1	7.80	7.70	0.987
	Mode 2	8.01	8.18	1.02
	Mode 3	8.46	9.19	1.09
Riverside MOB*	Mode 4	7.14	6.50	0.910
	Mode 5	7.46	7.88	1.06
	Mode 6	8.14	7.34	0.90
First Bank & Trust* Building	Mode 3	8.92	9.55	1.07
	Mode 5	10.6	8.81	0.831
			Average =	0.992
			COV =	7.05%

* Other measured and predicted natural modes did not have similar shapes.

Table 4.17 summarizes the measured and predicted peak acceleration responses to walking excitation. The predictions were made using response history analyses with individual footstep loading. The average ratio of predicted-to-measured peak acceleration is 1.47, indicating moderate over-prediction. Peak acceleration was slightly under-predicted in four of the fourteen cases and significantly over-predicted in six cases. The COV equals 38.6% indicating significant dispersion. The shaded ratios indicate that the accelerance FRF peak magnitude was significantly over-predicted. If these values are excluded, then the average ratio of predicted-to-measured peak acceleration is 1.18, indicating only a slight over-prediction. This indicates that inaccurate modal property prediction (mode shapes which directly affect the accelerance FRF magnitude and effective mass) is the primary cause for inaccurate predictions of maximum response to walking. The COV equals 23.8% if the shaded values are excluded, indicating moderate data dispersion.

Table 4.16: Summary of Accelerance Peak Magnitude Comparisons

		Accelerance Peak Magnitude		
Specimen	Description	Measured (%g/lbf)	Predicted (%g/lbf)	Predicted / Measured
Long Span Composite Slab Specimen	Mode 1	0.364	0.437	1.20
Long Span Composite Slab Mockup	Bay 1, Mode 1	0.153	0.165	1.08
	Bay 1, Mode 2	0.0680	0.166	2.44
	Bay 2, Mode 1	0.162	0.151	0.932
	Bay 2, Mode 2	0.0638	0.0961	1.51
Square-End Joist Footbridge	Mode 1	2.48	1.96	0.790
Shear-Connected Joist Footbridge	End Bay, Mode 1	1.57	1.46	0.930
	End Bay, Mode 2	2.03	1.35	0.665
	End Bay, Mode 3	0.313	0.516	1.65
	Middle Bay, Mode 1	1.37	1.42	1.04
	Middle Bay, Mode 3	1.55	2.08	1.34
Riverside MOB	Bay 1	0.151	0.153	1.01
	Bay 2	0.128	0.236	1.84
	Bay 3	0.118	0.116	0.983
	Bay 4	0.0770	0.192	2.49
First Bank & Trust Building	Bay 1	0.181	0.454	2.51
	Bay 2	0.261	0.407	1.56
	Bay 3	0.162	0.460	2.84
			Average =	1.49
			COV =	43.9%

Table 4.17: Summary of Walking Acceleration Response Comparisons (Predictions Using Individual Footsteps)

		Peak Acceleration Due to Walking		
Specimen	Description	Measured (%g)	Predicted (%g)	Predicted / Measured
Long Span Composite Slab Specimen	Parallel to Deck	1.36	1.68	1.24
	Perpendicular to Deck	0.972	1.68	1.73
Long Span Composite Slab Mockup	Bay 1	0.768	1.18	1.54
	Bay 2	0.966	0.790	0.818
Square-End Joist Footbridge	-	7.03	7.01	0.997
Shear-Connected Joist Footbridge	End Bay, Mode 1	3.32	3.06	0.922
	End Bay, Mode 2	4.14	4.24	1.02
	Middle Bay, Mode 1	2.64	3.43	1.30
	Middle Bay, Mode 3	4.67	10.2	2.18
Riverside MOB	Bay 1	1.45	1.34	0.924
	Bay 2	1.33	2.31	1.74
	Bay 3	1.03	1.33	1.29
	Bay 4	0.775	2.23	2.88
First Bank & Trust Building	Bay C-1/F-2	2.44	4.95	2.03
			Average =	1.47
			COV =	38.6%
			Average Excluding Shaded Values =	1.18
			COV Excluding Shaded Values =	23.8%

Table 4.18 summarizes the measured and predicted peak acceleration responses to walking excitation. The predictions were made using response history analyses with Fourier series loading. The average ratio of predicted-to-measured peak acceleration is 1.84, indicating significant over-prediction. Peak acceleration was very slightly under-predicted in one of the fourteen cases and significantly over-predicted in eleven. The

COV equals 27.2%, indicating moderate data dispersion. The shaded ratios indicate that the accelerance FRF peak magnitude was significantly over-predicted. If these values are excluded, then the average ratio of predicted-to-measured peak acceleration is 1.71, indicating significant over-prediction. The COV equals 28.1%, indicating moderate data dispersion. This method over-predicts the peak acceleration partially because the load is placed at mid-bay for the entire response history analysis, which was not the case during the measurements.

Table 4.18: Summary of Walking Acceleration Response Comparisons (Predictions Using Fourier Series)

		Peak Acceleration Due to Walking		
Specimen	Description	Measured (%g)	Predicted (%g)	Predicted / Measured
Long Span Composite Slab Specimen	Parallel to Deck	1.36	2.71	1.99
	Perpendicular to Deck	0.972	2.71	2.79
Long Span Composite Slab Mockup	Bay 1	0.768	1.38	1.80
	Bay 2	0.966	1.39	1.44
Square-End Joist Footbridge	-	7.03	11.3	1.61
Shear-Connected Joist Footbridge	End Bay, Mode 1	3.32	5.42	1.63
	End Bay, Mode 2	4.14	6.94	1.68
	Middle Bay, Mode 1	2.64	5.30	2.01
	Middle Bay, Mode 3	4.67	11.4	2.38
Riverside MOB	Bay 1	1.45	1.38	0.952
	Bay 2	1.33	2.28	1.71
	Bay 3	1.03	1.20	1.17
	Bay 4	0.775	2.06	2.66
First Bank & Trust Building	Bay C-1/F-2	2.44	4.28	1.75
			Average =	1.84
			COV =	27.2%
			Average Excluding Shaded Values =	1.71
			COV Excluding Shaded Values =	28.1%

Table 4.19 summarizes the measured and predicted peak acceleration responses to walking excitation using the simplified frequency domain method. The average ratio of predicted-to-measured peak acceleration is 1.71, indicating significant over-prediction. Peak acceleration was very slightly under-predicted in one of the fourteen cases and significantly over-predicted in eleven. The COV equals 29.2%, indicating moderate dispersion. The shaded ratios indicate that the accelerance FRF peak magnitude was significantly over-predicted. If these values are excluded, then the average ratio of predicted-to-measured peak acceleration is 1.58, still indicating significant over-prediction. If the shaded values are excluded, the COV equals 30.6%, indicating moderate dispersion. As is the case for the Fourier series loading, this method over-predicts the peak acceleration partially because the load is placed at mid-bay for the entire analysis, which was not the case during any of the measurements.

Because the simplified frequency domain method uses the accelerance magnitude, it provides an opportunity not possible when comparing results using other prediction methods: separation of the effect of modal properties and walking force.

Table 4.20 shows the predictions that would be made if the accelerance FRF magnitude was correctly predicted. The predictions shown in the Table were computed using the measured accelerance peak magnitudes, thereby eliminating the effect of incorrect FRF predictions. The average ratio of predicted-to-measured peak acceleration is 1.49, indicating moderate over-prediction. The response was significantly over-predicted in seven of the fourteen cases and moderately under-predicted in one case. The COV equals 34.1%, indicating moderate data dispersion. Interestingly, compared to the other floors, the response was very accurately predicted in all four of the Riverside MOB bays.

Table 4.19: Summary of Walking Acceleration Response Comparisons (Predictions Using Simplified Frequency Domain Procedure)

		Peak Acceleration Due to Walking		
Specimen	Description	Measured (%g)	Predicted (%g)	Predicted / Measured
Long Span Composite Slab Specimen	Parallel to Deck	1.36	2.59	1.90
	Perpendicular to Deck	0.972	2.59	2.66
Long Span Composite Slab Mockup	Bay 1	0.768	1.19	1.55
	Bay 2	0.966	1.06	1.10
Square-End Joist Footbridge	-	7.03	11.0	1.56
Shear-Connected Joist Footbridge	End Bay, Mode 1	3.32	5.13	1.55
	End Bay, Mode 2	4.14	6.92	1.67
	Middle Bay, Mode 1	2.64	4.98	1.89
	Middle Bay, Mode 3	4.67	11.2	2.40
Riverside MOB	Bay 1	1.45	1.38	0.952
	Bay 2	1.33	2.21	1.66
	Bay 3	1.03	1.02	0.990
	Bay 4	0.775	1.88	2.43
First Bank & Trust Building	Bay C-1/F-2	2.44	4.11	1.68
			Average =	1.71
			COV =	29.2%
			Average Excluding Shaded Values =	1.58
			COV Excluding Shaded Values =	30.6%

Table 4.20: Summary of Walking Acceleration Response Comparisons (Predictions Using Simplified Frequency Domain Method Using Measured Accelerance FRF)

		Peak Acceleration Due to Walking		
Specimen	Description	Measured (%g)	Predicted (%g)	Predicted / Measured
Long Span Composite Slab Specimen	Parallel to Deck	1.36	2.11	1.55
	Perpendicular to Deck	0.972	2.11	2.17
Long Span Composite Slab Mockup	Bay 1	0.768	1.12	1.46
	Bay 2	0.966	1.30	1.35
Square-End Joist Footbridge	-	7.03	14.0	1.99
Shear-Connected Joist Footbridge	End Bay, Mode 1	3.32	5.60	1.69
	End Bay, Mode 2	4.14	10.3	2.49
	Middle Bay, Mode 1	2.64	4.88	1.85
	Middle Bay, Mode 3	4.67	7.77	1.66
Riverside MOB	Bay 1	1.45	1.52	1.05
	Bay 2	1.33	1.28	0.962
	Bay 3	1.03	1.05	1.02
	Bay 4	0.775	0.718	0.927
First Bank & Trust Building	Bay C-1/F-2	2.44	1.66	0.680
			Average =	1.49
			COV =	34.1%