THE DESIGN, DEVELOPMENT, AND IMPLEMENTATION OF A STRUCTURAL DYNAMIC RESEARCH LABORATORY

By

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A THESIS PRESENTED TO THE GRADUATE SCHOOL
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The thesis is dedicated to my parents for their
continual love and support throughout my academic career

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Advancing research in the area of structural dynamics results in safer and more efficient structures. A broad range of approaches to addressing the challenges faced by structures subjected to dynamic loads exist in both analytical and experimental domains. Experimental structural dynamics research enables enhanced understanding of structural and component response to dynamic loading conditions and aids in the creation of accurate structural models. To conduct meaningful and informative structural dynamics research, the appropriate equipment and structural models must be developed and implemented.

The research presented in this thesis is focused on the creation of a comprehensive and versatile structural dynamics laboratory at the University of Florida. This research consists of multiple project phases that include planning, design, and implementation to provide a functional platform to perform structural dynamic experiments. The work is separated into two distinct components that are necessary in a structural dynamic research laboratory: excitation equipment and structural models. Prompt installation of a preassembled uniaxial shake table and long stroke shaker provided experimental potential for small scale research applications. Documentation is

also provided for the extensive installation, assembly, and operational process of a larger scale six degree of freedom hydraulic shake table. Structural models included in this work consist of a three dimensional steel truss bridge, a small scale two-dimensional building model, and a larger scale three-dimensional steel building. The development of each of these models had similar project phases, including: design, finite element modeling, fabrication, and dynamic testing to quantify dynamic properties. The final outcome of this research is a laboratory facility with three calibrated structural models representing a number of structural forms with the flexibility for modification to support a variety of experimental investigations and three dynamic excitation instruments with a range of capabilities and configurations to enable testing of many sizes and forms of structural models, components, and sensors.

CHAPTER 1
INTRODUCTION

Since the early stages of structural design, society has relied heavily on an engineer's ability to predict the response of a structure under a given loading condition and ultimately assign a safe and dependable design. Over time, increasing demand for taller buildings and longer bridges, while minimizing costs and preserving aesthetics, has transformed this valued skill into an essential responsibility. Modern design methods predominantly rely on sophisticated modeling software and advanced analysis techniques, which have been improved through years of experimental testing and research in many engineering disciplines such as structural dynamics.

Structural dynamics can be defined as the behavior of a structure under dynamic or 'time-varying' loads with subsequent dynamic analyses being applied to find fundamental dynamic properties. Structures are arbitrarily subjected to dynamic loading as a result of:

- Pedestrian activity
- Structural member collapse
- Oscillating machinery
- Earthquakes
- Water impact
- Wind
- Explosive blasts
- Vehicular traffic

With progressions in computational capabilities and software packages, dynamic analysis methods have shown significant improvement since the 1970s (Tedesco et al. 1999). Despite these advancements, experimental evaluation of structural performance remains a critical area to improve structural design and public safety.

Research in structural dynamics is growing as new testing capabilities and loading simulations are made available. Although most publicized, earthquake forces are not the only excitations being replicated for dynamic experiments. Wind engineering

is an area in structural dynamics research that has recently improved both its facilities and testing capabilities and studies the structural response to wind loading. High powered turbines can produce wind loads simulating a hurricane (Shen et al. 2013 & Masters et al. 2008) or tornado (Haan Jr. et al. 2008) event at full-scale. Blast or shock load testing is another type of dynamic experiment. With the use of a dynamic drop hammer, the testing facility in Millard et al. (2010) is able to conduct blast simulations and provide appropriate design recommendations of structural members subject to juggernaut type impact. Ongoing research in these fields can help improve the performance of a structure during and after an extreme loading condition as well as provide critical information to improve future structural designs.

Motivation

While governing codes ensure the quality of a structure's design under typical loading conditions, the continual revision and expansion of provisions involving design for seismic, wind and other dynamic loads proves there are still many areas of structural design that can be studied and improved. Overstressing and collapse of structures coupled with extensive cracking are both undesirable effects of dynamic loads. Earthquakes in particular, known for unpredictable and forceful behavior, have yielded catastrophic effects to infrastructure across the world such as in Chile and Japan where Richter scale magnitudes reached 9.5 and 9.0, respectively (USGS 2012). Shown in Figure 1-1 is an example of the resulting damage earthquakes can incur.

Figure 1-1. Building response to earthquake in Chile (Image taken from: Tweedy, J. (2010, March 1). Chile earthquake: Santiago airport remains closed as tour operators cancel holidays. Mail Online. [Photograph]. Retrieved from https://www.dailymail.co.uk

Continued research in structural dynamics can lead to new and progressive designs intended to absorb high strain energy levels and ultimately prevent the total collapse of a structure and subsequent loss of life.

A structural dynamics research facility enables the recreation of multiple types of excitations in a controlled environment and the collection and analysis of resulting structural response data. The complexity of a dynamic research facility can vary significantly as it is directly dependent on a few major factors that will be briefly identified here and discussed in later detail.

The most critical component of a structural dynamics research facility is the excitation equipment providing the ability to produce and impose appropriate dynamic loads. This equipment varies in application, system type, and is capable of generating an array of excitations for structural dynamics research.

Also of importance in such a research facility are the experimental structural models. These full or small-scaled structures are valuable in addressing a structural system's response to a given loading condition, resulting in potential improvements to many aspects of structural engineering.

Finally, without the proper data collection devices such as accelerometers, strain gauges, radars, or LVDT's, researchers are unable to report any sort of detailed system response or structure identification. These sensing and data acquisition capabilities can provide fundamental knowledge of dynamic structural properties such as natural frequencies, mode shapes, and damping ratios that can be used to understand both linear and nonlinear dynamic behavior.

With appropriate post processing of data obtained from dynamic experiments, other structural dynamic effects can also be determined. For example, experimental dynamic deflections on a scaled model can be obtained to verify serviceability requirements will be met (De Wilde et al. 2010). The E-Defense shake table, discussed in Chapter 2, was used by Ji et al. (2011) to investigate the effects of seismic damage on a specimen representative of a high-rise building by imposing vibrations, measuring response data, and applying damage diagnosis methods. System identification, a method applied to examine the inherent properties of a structure, was completed in the dynamic tests conducted by Astroza et al. (2013) using various excitation methods such as ambient, impact, and seismic vibrations. Lastly, work done in Rice and Spencer (2008) showed structural response is not the only area of interest in dynamic testing. A test structure with a known response was used in conjunction with wired accelerometers as a baseline measurement to develop a new wireless sensor platform that can collect vibration and temperature data for structural health monitoring applications.

The objective of this work is to produce a structural dynamics research laboratory capable of performing numerous types of dynamic experiments while maintaining accuracy of measured results by assembling the equipment and models in the facility in

a systematic and calibrated approach. This research includes supporting information and detailed documentation on two critical components necessary in a dynamic research lab: (1) excitation equipment and (2) structural models. This work will provide a general guideline for future structural engineering researchers focusing on structural dynamic experiments.

Content Overview

This section outlines the contents of the research presented in this document. Chapter 2 provides a general background on structural dynamics and reviews different cases of related work. Chapter 2 is separated into three discrete sections: dynamic forces, structural models, and excitation equipment. Selected dynamic research facilities are compared and discussed with a focus on their experimental equipment and dynamic testing capabilities. Structural models, small and full-scale, are presented outlining their functionality, benefits and limitations, and their importance to understanding dynamic response. Additionally, the capabilities and performance expectations of specific categories of shaker devices are compared.

Chapter 3 summarizes the installation process, research potential, and specifications of two types of excitation equipment used in the lab; smaller, “lab ready” devices that require the basic installation and understanding of controls, and larger, component based systems where distinct installation methods are required.

Chapter 4 outlines three unique structural dynamic research models in detail including the preliminary design and analysis, construction, assembly, and experimental suitability.

Concluding remarks summarizing the work is presented in Chapter 5, where future work and recommendations are also discussed.

CHAPTER 2
BACKGROUND AND RELATED WORK

To perform dynamic tests with the confidence that data results are valid and reportable, the design goals of a structural dynamics laboratory must be established and satisfied. The beginning of this chapter will provide a general background on structural dynamics as a reference. Subsequently, a discussion on a variety of structural models and their significance in structural dynamic applications as well as a comparison of different types of shaker systems is presented.

Background

To describe how the mass of a system significantly influences the dynamic response, the following set of equations (Equation 2-1 to 2-3) are simplified relationships describing a single degree of freedom dynamic system. The natural circular frequency, ω, of a single degree-of-freedom (DOF) system, measured in radians/second, is a direct function of two system properties; mass (m) and stiffness (k) and can be described by the following equation:

$$\omega = \sqrt{\frac{k}{m}} \tag{2-1}$$

Shown in Equation 2-2, the inverse of the natural frequency is the natural period (T), which is the number of seconds it takes for a system to go through one full oscillation.

$$T = \frac{2\pi}{\omega} \tag{2-2}$$

This parameter can also be manipulated to yield another common quantity, the natural frequency (f), which is the number of cycles a single DOF system oscillates in one second:

$$f = \frac{1}{T} = \frac{\omega}{2\pi} \tag{2-3}$$

To estimate dynamic response, the infinite DOF of a physical structure can be reduced based on how the majority of the mass is distributed within a system. This generally occurs at each floor level of a building. The new number of DOFs, n, can be directly applied to a dynamic analysis, describing a structural system of order n. The linear motion of a multiple DOF structure can then be idealized as a lumped mass model and is described by the following equation:

$$\mathbf{M}u'' + \mathbf{C}u' + \mathbf{K}u = F(t) \tag{2-4}$$

where $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ represent respectively the mass, damping, and stiffness matrices of size n x n. The variables u'', u', u, and $F(t)$ represent respectively the acceleration, velocity, displacement and force vectors of order n. In dynamic experiments, $F(t)$ is the time-varying external force applied to a structure.

In the experimental approach presented in this work, vibration forces are applied to a test structure and subsequent modal analysis is conducted to obtain unique dynamic characteristics called “modes”. Modes can be defined by two components, a modal frequency value and a mode shape, both of which are influenced by a number of design factors such as mass, stiffness, damping and boundary conditions. Mode shapes are specific patterns at which a structure will vibrate when exposed to a natural frequency.

Resonant vibration occurs when one or more of the modal, or natural, frequencies of a structure are excited, resulting in the amplification of deflections, stresses, and strains to a level far beyond those caused by static loading. Incorporating a “resonance search” as a pretesting method is a controlled and reliable procedure to

determine the natural frequencies of a new structural model. A resonance search is conducted at lower excitation intensity than main tests to sustain minimal damage potential to a test structure.

Detailed modal analysis procedures involving matrix manipulation and linear algebra applications have been thoroughly documented (Mota 2011, Rezai 1999, and Schwarz & Richardson 1999); however, the general concept is briefly described here. Shown in Figure 2-1 is structural response data plotted in the frequency domain.

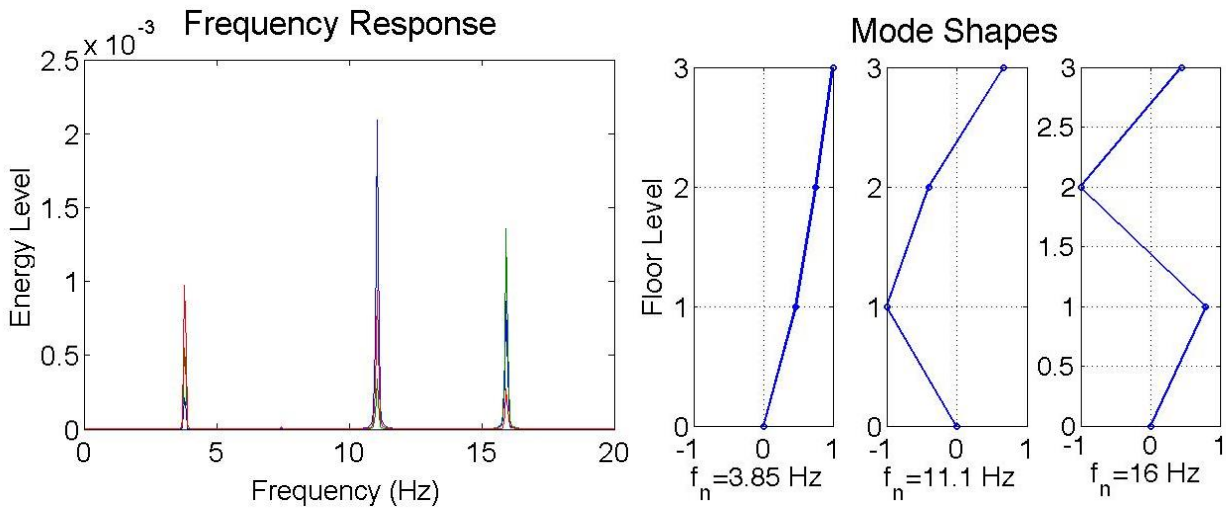


Figure 2-1. Frequency response and corresponding mode shapes

A power spectral density (PSD) function is a Fourier transform function which transforms experimental time history response data into the frequency domain to quantify how much, and at what frequency, the applied energy is being distributed throughout a system. By applying a structural identification method known as “peak picking”, the natural frequencies can be visually estimated according to where dominating peaks fall in the frequency domain plot. Using estimated natural frequency values and analyzing the phase relationship between data collected from selected sensors, the identification of corresponding structural mode shapes can be plotted

(Figure 2-1). For the example shown Figure 2-1 for Mode 1, all sensors are estimated to be in phase with one another and displace in the same direction when excited at a frequency of 3.85Hz.

Many other modal analysis methods have been developed and are used in structural dynamic research for detailed structural identification applications. Each has their own benefits and limitations depending on the structural model, loading conditions and sensor capabilities. The modal identification presented in this work mainly relies on the peak picking method. However, other means for system identification are available such as the Eigensystem realization algorithm (Qin et al. 2001).

Dynamic Forces

To better understand a structures' response to dynamic effects, methodologies for dynamic testing must be developed. Attempts to capture full-scale, in-service structural response data during a seismic loading event may be achieved, however it is challenging, if not impossible, to predict when this arbitrary load will occur. Simulation of dynamic loading conditions on a structure within a controlled laboratory setting is a convenient advantage of a dynamic research facility, allowing for complete control of when and how often a test specimen will experience a loading event. This ability to generate dynamic loading scenarios can vary depending on the scope of structural dynamic research and the limitations of shaking equipment.

Forced Vibration

The forced vibration testing methods identified in this section can be applied to a system by use of an excitation device. These versatile pieces of excitation equipment, explained in detail in the last section of this chapter, have multiple modes of operation and have the potential to input energy into a structural system in a variety of different

ways. Highlighted in this section are three types of signals for which forced vibration can be applied: (1) known harmonic signals intended to excite a discrete frequency, (2) stochastically generated signals where the applied motion is defined by its statistics, and (3) signals representing physical phenomena such as an earthquake.

Generated signals where input energy is artificially manipulated to allow for complete control of testing parameters is a common forced vibration testing method for the initial stages of a testing program. Here input excitations can be applied at known energy levels and frequencies as a conservative approach to dynamically characterize a system. The primary motivation for this type of forced vibration test is that if loads applied to a test structure are known and the response can be monitored and recorded, then structural dynamic properties are able to be estimated. Simple periodic wave motions such as sine, square, saw tooth and triangle, are implemented and can be modified or amplified accordingly to fit the test application. A well-documented and widely used resonance search technique involving a more complex version of a sine wave motion is a “sine sweep”. Sinusoidal sweep testing generates a periodic sine wave motion at an initial frequency value that increases incrementally to an upper frequency limit and can vary as a function of sweep velocity. This is applied so each natural frequency of a structure in the selected interval will be excited causing resonance and the visual identification of resonant frequencies (Gloth & Sinapius 2004). Shield et al. (2001) applied a sine sweep to identify system characteristics of a large-scale single degree of freedom system before submitting the test structure to seismic simulations.

In randomly generated signals, the only parameters under the control of a testing engineer are the desired statistics of the signal, resulting in an anticipated root mean square of the amplitude of the signal. This type of testing is stochastic and is more representative of practical loading conditions such as vehicular traffic on a bridge, wind forces on a building or other projected service loads. Therefore, this method is proven as an attractive avenue for in-service time vibration monitoring of real structures under normal service loads (Salawu and Williams 1993) and allows a test structure to be excited with energy at all levels within a specified frequency range. This interval can be created with a wide range of low and high cutoff off frequencies, however applying this signal onto a structure largely depends on the capabilities of the excitation equipment. One or more shakers are used for excitation and are oriented in a variety of ways, both depending on size and complexity of a test object and the vibration modes of interest.

In earthquake-prone regions, a fundamental concern of structural engineers is the behavior of structures exposed to earthquake-induced motion. As a result, one of the most important applications of structural dynamic studies is analyzing the response of structures to ground shaking caused by an earthquake. In this case, forced vibration is not used for finding the resonant frequencies of a structure; instead they are vital to the application of earthquake time histories and observation of the models' response. Earthquake time histories are actual records of past earthquakes that have been recorded using a strong motion accelerograph to capture motion in the x-, y-, and z-directions. Shown in Figure 2-2 is an acceleration-time record of the El Centro earthquake in 1940.

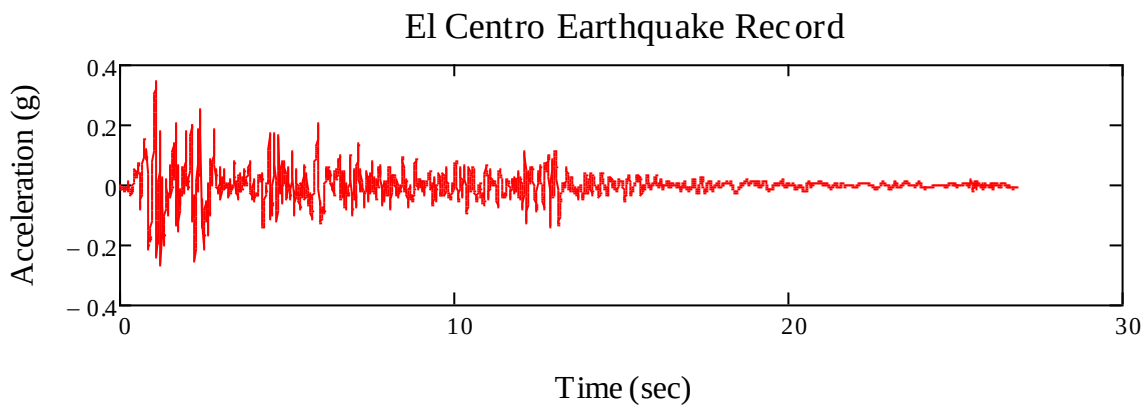


Figure 2-2. El Centro Earthquake Time History

Resulting ground accelerations can be incorporated to the equation of motion as an applied or external force; shown below:

$$\mathbf{M}u''+\mathbf{C}u'+\mathbf{K}u=-\mathbf{M}u_g''(t) \tag{2-5}$$

This is the equation of motion of a structure subjected to ground acceleration $u_g''(t)$.

Given that an earthquake's intensity, duration, and frequency content all depend on independent parameters such as environmental conditions, proximity to fault lines, and seismic wave patterns, each earthquake possesses thumbprint like qualities that result in no two records being the same. As such, the comparison of unique earthquake characteristics from multiple time histories has led to substantial findings which classify ground motion into four basic types (Newmark and Rosenblueth 1971); single-shock, moderately long and extremely irregular motion, long ground motion with distinct dominant periods of vibration, and large-scale permanent ground deformation. Three earthquake characteristics established from basic ground motion classifications and found to significantly influence structural response are identified as peak ground motion, duration of motion, and frequency content. Peak ground motion transformed to peak

ground acceleration (PGA) can be used in dynamic experiments for the scaling and application of acceleration time histories on structural dynamic models.

Impact

Impulsive excitation methods can be adopted for dynamic experiments relying solely on free vibration coupled with initial displacement or velocity of a structure. The basic concept involves the use of an impact (short duration force) to apply energy to a structure in order to excite the mass and cause the system to accelerate. Accelerations are then measured with accelerometers that are strategically placed at particular degrees of freedom. This can be a quick, simple and low cost approach for finding the modal properties of a test structure, however like most methods, input energy limitations are introduced when larger scale test specimens are of interest. Forces can be generated by an impact hammer either from manual arm motion or a drop hammer apparatus. Hammer size variations depend on the both the type and scale of a test specimen and must provide appropriate impact force to generate measurable vibration levels after impact. Lynch (2006) used a modal hammer to impose energy on a full-scale bridge to verify the functionality of a new wireless sensor. The hammer size and location of impact yielded reasonably high acceleration responses that were able to be recorded and post processed to compare experimental and known natural frequency data. Although most simple, the impact testing method is highly susceptible to input noise due to the short impact period when force is applied. Also, test procedures must be carefully followed as the human error element is nearly unavoidable. For example, not allowing the impacted structure to come to a rest in between hammer blows could significantly magnify or nullify a structure's vibration response. Additionally, upon impact, there is a high potential for a "double hit" which could alter response data

requiring a retest. Because of these test flaws, it is essential that a modal hammer is equipped with a load cell at the impact tip; this facilitates direct measurement of imposed force to assist in system post data processing and confirm tests were conducted with minimal error.

Comparison of Methods

As presented in this section, there are a variety of methods for imposing excitation on a structure. Each method has ideal applications and drawbacks. Compared in Table 2-1 are a few testing characteristics of the two excitation methods discussed in this section.

Table 2-1. Compared Excitation Methods

Testing Characteristics	Excitation Method		
	Forced Vibration		Impact Hammer
	Steady State Sine	Random	
Signal to Noise Ratio	Very High	Fair	Not good
Test time	Very Long	Good	Very Good
Controlled Frequency	Yes	Yes	No
Controlled Amplitude	Yes	No	No

Signal to noise ratio, test time, and the ability to control input frequency and signal amplitude are all factors that must be considered when designing a structural dynamic testing program. With varied differences in testing capabilities, an experiment can be conducted with the most suitable excitation method.

Reynolds and Pavic (2000) compares the functionality of impact hammer and mechanical shaker excitation methods for modal testing of a full-scale building floor.

Figure 2-3 and Figure 2-4 show the shaker system (APS Dynamics Model 113 “Electro-Seis” electrodynamic shaker) and the instrumented impact hammer (Dytran Model 5803A) compared in this study.

Figure 2-3. Shaker System (Image taken from: APS Dynamics. n.d. *APS 113 Electro-seis*. [Photograph]. Retrieved from http://www.apsdynamics.com. October 1,2013.)

Figure 2-4. Impact Hammer (Image taken from: Dytran Instruments, Inc. n.d. *Impulse Hammers*. [Photograph]. Retrieved from http://www.dytran.com. October 1, 2013.)

Four key factors used for practical comparison of a widely used shaker system and impact hammer are listed below:

1. Initial equipment cost
2. Time and resources needed to develop the test system
3. Ease of set up and implementation
4. Quality of measured data

Evidence from this comparative study found that the cost of implementing a modal test system for examining the vibration behavior of a building floor is ten times greater for a shaker excitation system as compared to hammer impact excitation. This is due to the coupling of the required power amplifier, reaction mass assembly, digital signal generator and accessories for a shaker system. Furthermore, development time was minimal for the impact hammer compared to the shaker systems' elaborate steps and time demands needed to understand, generate, process, and amplify a given signal. Conversely, the shaker system proved much more reliable when comparing data quality due to the control of input parameters and ability to strategically manipulate a motion or range of signal frequencies. For research applications where expenses are not always a limiting factor and quality of data prevails as the most critical component to an experiment, it is suggested that a shaker excitation system be used. Specifically, electrodynamic shaker systems are one of three shaker systems covered in the "Excitation Equipment Section" of this chapter. Where financing options are limited and only a quick baseline test of a structure is needed then implementing an impact hammer proves to be the best value of money (Reynolds and Pavic 2000).

Structural Models

Experimental testing of structural models in a dynamic research facility provides a wealth of information to improve understanding of structural dynamic behavior. There are numerous advantages these models can provide that continually prove their value in dynamic research and engineering industry. A few of these benefits include:

- The validation or improvement of structural design methods, procedures, or new structural systems that can be considered for future governing code revisions or practice techniques

- The accurate correlation between finite element predictions and actual system response
- The development of new instrumentation devices such as displacement or acceleration sensors

With a versatile range of applications, structural models range in size, type and system complexity. Considering that dynamic testing facilities have limitations on equipment and power supply options, it is common to find simplified experimental structures using lumped mass models that consist of smaller scales and point mass distributions. These simplifications provide a functional structural model for which to identify dynamic characteristics and preserve the dynamic response of a full-scale structure. With the increase in excitation equipment system capacity, structural models can be designed on a large or even full-scale to be geometrically representative of a real structure. Illustrated in Figure 2-5 are three main categories for which structural models can be applied to structural dynamic applications. This section will focus on two of these categories, highlighting previous related applications and significance of both structural systems and structural members.

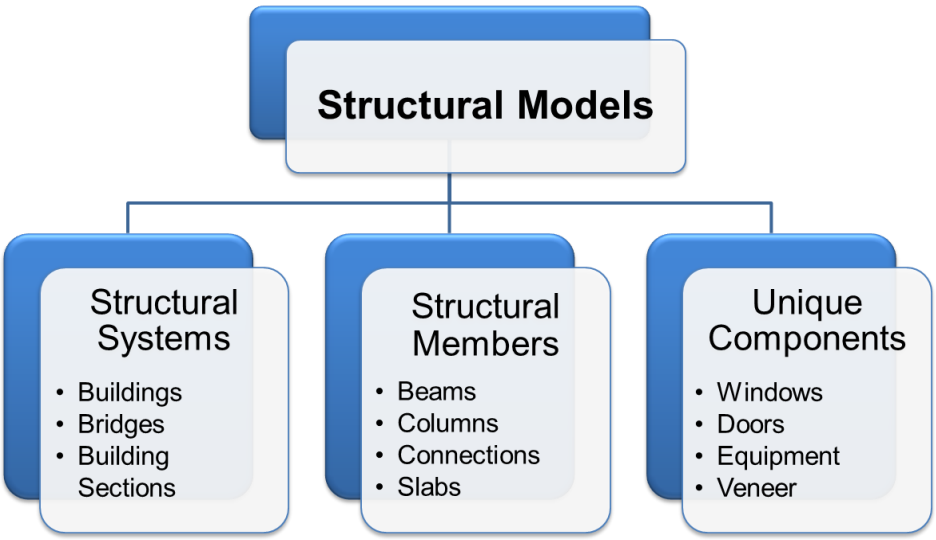


Figure 2-5 General Structural Model Types

Structural Systems

The design and development of an experimental structure that models a parent structural system can be an effective tool in a testing program to predict the response to dynamic loading. In both small and full-scale applications, these systems can incorporate a collection of beams, columns, slabs, or connection types to collectively represent a real building, bridge or other structural system. For instance, a full-scale, light-frame wood building was used to measure the response due to a seismic load applied by a shake table (Van de Lindt et al. 2010). This model allowed for the measurement of interstory drift as well as shear wall deformations at multiple excitation levels in order to verify the accuracy of predictions made from a SAPWood finite element model.

As previously mentioned, decisions on how large or small a test structure is to be designed is largely dependent on a laboratories' equipment capabilities and test budget constraints. Rather than constructing an entire building model, Dolce et al. (2005) implemented a single building section model to estimate a full-system response to dynamic loads. A single 1/3 scale reinforced concrete frame was used in conjunction with a shake table to test the effects of various energy dissipating braces in order to enhance seismic performance. The 1/3 scale was selected to allow for full exploitation of the dimensions and payload capacity of the shake table system. Directly relating to wind loads, a similar sectional approach was used in (Hanson et al. 2000) where a bridge girder section was subject to wind tunnel tests to predict the response of a parent bridge to wind force and direction variances.

Structural Members

Rather than studying a global system response of a structure, isolated structural members can be subjected to dynamic testing to analyze the behavior of individual beams, columns, or walls. Designing a test program to study the performance of a specific area of a structural system where weaknesses have been known to occur not only isolates a specific test area of interest, but also significantly decreases the material and fabrication cost of a test specimen as well as overall complexity of an experiment.

Often, the introduction of a new structural product to the design industry requires research aimed at providing an understanding of how it will respond to specific loading scenarios. By designing a dynamic fatigue test at a simpler structural member level, Zaghi et al. (2012) was able to investigate the increased ductility potential of a fiber reinforced polymer (FRP) concrete column under dynamic loading conditions. Quicker fabrication time of a structural member, as compared to an entire structural system, can result in prompt recommendations such as the viability of FRP for retrofit applications. Similarly, structural member test specimens can be used to validate or improve design methods and capacities anticipated by code procedures. Testing of an unreinforced retaining wall by Ling et al. (2005), found the lateral load capacities predicted by a conventional design method could be increased.

Full-Scale vs. Small-Scale

Structural models are versatile tools used in multiple structural dynamic research areas; however, model size and experimental applications are often limited to the capabilities provided by the laboratory. As a result, determining an appropriate structural model size becomes an obstacle as there are many benefits and drawbacks to having a model at opposite ends of the spectrum.

Full-scale models replicate the material and makeup of a real structure or structural member and are as close to their full size and boundary conditions as possible. Kasai et al. (2010) used a full-scale, 15.8-meter, five story steel building as a test specimen on the E-Defense shake table in Japan. Focusing on structural component development, the performance of four types of dampers were monitored and evaluated by imposing full-scale earthquake forces to the structure.

Small-scale models, idealized structural systems on a much smaller scale, can be used in testing environments where full-scale testing is not capable. Work done by Wu and Samali (2002) involved a five story steel frame model used as a benchmark structure to allow researchers the ability to test control algorithms. Standing at a maximum height of three meters, this test specimen offers the flexibility needed to model and test various building configurations. Replaceable beam-column connectors allow for connections to be interchanged from pin to fixed conditions. By adding plates or point masses at pre-designated locations, the total mass of the model can range between one and three tons. Additionally, an innovative joint design facilitates floor height modification for each of the five stories.

Both small and full-scale models have proven their importance to structural dynamic research on many occasions and are evaluated below using similar key points for comparison as Reynolds and Pavic (2000) identified earlier in the chapter.

Cost, time and resources. When evaluating small and large-scale models, it is apparent that larger models almost always result in increased expenses. Labor costs become a significant factor in a large-scale project budget due to an overall increase in fabrication time, processes, and planning in the design and construction of a model.

Small-scale applications serve as a simplified approach to dynamic testing with very few components included in design. Structural members tend to be symmetrical in shape and dimensioned for ease of construction which directly reduces the model material and fabrication cost. With the lower cost and prompt fabrication time of these small-scale models, sacrificing a test specimen to obtain critical data on a particular limit state can also be an allowable option and unique advantage. Aside from the models themselves, the excitation equipment needed to reproduce the dynamic loads of interest add to the overall expense of a test setup. Shake table systems for example, require the installation of an adequately sized power supply and a designated facility to contain experiments and protect the functionality of the equipment. With the demands of a full-scale experiment, associated costs can grow to hundreds of millions of dollars (Kallinikidou 2004). As a result, there are only a select few testing facilities around the nation and worldwide capable of performing these types of experiments, most of which have been adopted into the Network for Earthquake Engineering Simulation (NEES) program which is discussed in detail in the next section

Quality of measured data. The design of any structural model, numerical or experimental, incorporates a number of critical assumptions and appropriate simplifications. Parameters such as boundary conditions, material properties, and applied loads are all areas of uncertainty in the design of these research tools. In full-scale models, these assumptions are minimized because of the replicated design and construction of a real structural system or member. Therefore, full-scale models can be a valuable dynamic testing tool when a structural system of interest is complex and if difficulty arises when trying to rely solely on small-scale testing or computer models to

predict system response (Salawu and Williams 1993). For example, if a concrete building model is used for dynamic testing, the yield capacity of the steel reinforcement, the compressive strength of concrete, and the construction phases can be practically conserved. By replicating these processes, the data obtained from such experiments is expected to closely represent the true response of the real structure. Although it has been proven that reliable conclusions can be drawn from small-scale structural model testing (Abrams 1992), data validity is largely influenced by scaling effects that may alter results or make data interpretation a less direct procedure. Scaling factors can be determined by “similitude laws” that have been studied and implemented as a widely accepted approach to the design of test specimens. Further discussion on the effects of similitude relationships are presented further in the “Similitude” section of this chapter.

Structural dynamic research interests range in diversity and can include many types of experimental set ups of shaker and structural model combinations. Critical data about a structure can be obtained from both small and full-scale systems. Where funding and space limitations are no concern, full-scale testing provides the most accurate test conditions. However, the ability to carry out full-scale tests is not practical in most cases; and small and large-scale structural model variations have been widely accepted in most dynamic testing facilities.

Similitude

Due to restrictions on space and equipment, as well as economic constraints, most experiments do not utilize full-scale models. Instead, structural members, material, connections, and loads are scaled down to a practical level that can be functional in a laboratory setting. This scaling, or similitude, is introduced to structural

dynamic testing as a systematic approach to scale a model and its applied loads to create a test set up that closely resembles a full-scale structure.

The details of similitude theory may be found in literature (eg: Rezai 1999 and Mota 2011); however a general description is briefly provided here. The fundamental physical measures most commonly used in engineering applications are mass (M), force (F), length (L), and time (T). These basic quantities are combined to yield more detailed units of measure such as acceleration or pressure. Using dimensional analysis techniques (Moncarz & Krawinkler, 1981), relationships between a full-scale system and its model can be made and appropriate scaling values assigned.

In dynamic similitude, it is common to use length (L), modulus of elasticity (E), and acceleration due to gravity (g) as the fundamental units of measure to derive scaling relationships. In experiments involving reduced scale models in which length is scaled, force and time need to be scaled as well to simulate an equivalent loading condition on the scaled structure. Some common and established relationships (Krawinkler 1988) used in the design of scaled models are listed in Table 2-2 and explained below:

Table 2-2 Similitude Scale Factors

Physical Quantity	Scale Factor
Length	$1/L_r$
Time	$1/\sqrt{L_r}$
Frequency	$\sqrt{L_r}$
Force	$1/L_r^2$

Scaling factors are typically expressed as ratios and relate a unit of measurement from the full-scale (L_p) to the model (L_r). For example, a quarter-scale model (L_p/L_r = ¼) will

have a length scale factor of $L_r = 4$ and denotes one unit of length of the prototype corresponds to four units of length for the smaller model. To apply an earthquake time-history to this model, the time scale is scaled by $1/\sqrt{L_r} = 1/\sqrt{4} = 1/2$ and the frequency content scaled by $\sqrt{L_r} = \sqrt{4} = 2$. Naturally, by decreasing the duration of a time history, the frequency content will be increased, and this is illustrated by the scales presented above. Despite some early critics regarding the accuracy of these scaling factors, the reliability of experimental results of one-half to one-third scale models have been generally accepted by many researchers since the 1980s (Rezai 1999).

Applying similitude relationships can vary depending on the experimental data of interest. For instance, if dynamic response of a small-scale model was to be measured, then the appropriate mass distribution and boundary conditions must be applied to conserve the dynamic similitude of the full-scale parent structure. In other cases, model designs can incorporate intricate detail to study localized strength limit states Lu et al. (2008) used a detailed similitude procedure to scale down a 33-story reinforced concrete building to a 1/25 building model. In this instance, fine-aggregate concrete reinforced with thin steel wires were used in an effort to maintain a legitimate level of material similitude when scaling a reinforced concrete prototype. The fine aggregate, 1.5mm in size, and reinforcement with minimal diameter, were chosen to match dictated similitude requirements as best as possible. Years following, a full-scale dynamic test was completed on the in-service structure upon its erection using ambient wind loads and the primary excitation method. It was concluded that the previous shake table tests performed at a 1/25 scale were reasonable and similar small-scale tests can be used in the future to help meet design practice needs.

In contrast to the work completed by Lu et al. (2008), there are simpler structural models that do not preserve intricate component detail use to study global dynamic response. More idealized systems like a small-scale steel test structure representative of a mid-rise steel building (Chen & Chen 2002 and Wu & Zamali 2002) can be just as informative with the proper post processing and modal analysis of response data.

Instrumentation

The general components that make up a dynamic research facility have been identified and include the ability to generate dynamic loads and the implementation of test structures. In addition to these components a test facility must have the ability to measure the input excitation and a range of structural model responses by the use of sensing devices. Essential to structural dynamic research, sensors provide an avenue for researchers to collect and analyze experimental data and gain a thorough understanding of a structural system or material type.

Sensor types. Sensing devices can reside in one of two groups, active, where external electrical energy is needed for the device to function or passive, where self-contained energy sources are present and no external activation is required (De Silva 2007). Table 2-2 lists some of the sensor types used in structural dynamic applications along with their functionality.

Table 2-3 Sensor Types and Functionality

Sensors	Measurement
Strain Gauges	Material Strain
Accelerometers	Accelerations
Tachometer	Velocity
LVDT	Displacement
String Potentiometer	Displacement
Load Cell	Force
Barometer	Pressure

There are many different types of sensors whose application is directly influenced by the desired data of interest. An instrumentation device passes through two stages while making a measurement; first the physical variable being measured is sensed and converted to an analog, then the signal is converted into a form (i.e. digitized) that is suitable for a signal conditioning and processing.

In some dynamic experiments, velocity and displacement are both determined by directly integrating acceleration response data. In instances where noise may lead to flawed results, it is good practice to use separate sensors to measure velocity and displacement directly and for cleaner data to be obtained. It is not common, however, to differentiate a displacement measurement to determine velocity or acceleration due to the amplification of any noise present in response data. Of the sensor types listed above, two will be briefly identified: accelerometers and strain gauges.

Accelerometers are common sensing devices that measure the acceleration of an object. A variety of sensing mechanisms are utilized in accelerometers, with each suited to specific measurement condition. Some accelerometers are not capable of measuring very low frequency or DC vibrations, while others possess high sensitivity for measuring very low-amplitude vibrations. Trade-offs in cost, physical size, measurement bandwidth, sensitivity, and noise levels must be considered when selecting the most appropriate sensor for the experimental application.

When material strain relationships are an area of focus, strain gauge variations are a widely adopted and an inexpensive sensing device. By measuring strain, engineers are able to quantify the magnitude of tension or compression a member is experiencing and resulting studies on material fatigue or deformation can be conducted.

Strain gauge devices function by measuring the change in electrical resistance across the strain gauges as it undergoes a change in length. Strain gauge selection is based on the material being tested and whether the gauges will be surface mounted or embedded. Arrays of strain gauges in “rosette” patterns have the ability to provide principal stress information. The primary challenges associated with the use of strain gauges are their potential susceptibility to measurement noise and their measurement variability in changing temperatures. These factors must be considered and mitigated before and during testing.

Sensor placement. Sensor placement on an experimental model depends on numerous factors including: limitations in the number of available sensors/data acquisition channels, the type of analysis to be conducted (static or dynamic), and the expected response of the structure (elastic or inelastic). As a result, strategic sensor placement is required to allow for appropriate data to be collected while making efficient use of sensors devices.

Figure 2-6 is an example sensor layout on a section of a small-scale steel structure with steel plate shear walls subjected to earthquake vibrations by a shake table (Rezai 1999). A total of five separate sensor types were deployed on the test structure based on expected stress concentrations, available data acquisition channels, and nodal locations. Accelerometers were placed at each story level, or nodal location, to measure absolute accelerations with an additional four at the top floor to monitor the torsional response of the structure. In addition to acceleration, string-pot displacement transducers were installed at each floor level to capture in-plane displacement data. Strain gauge locations were selected where large deformations were expected to occur.

This is illustrated by the higher concentration of strain gauges coupled with the additional string-pots toward the bottom of the structure where stresses are greatest.

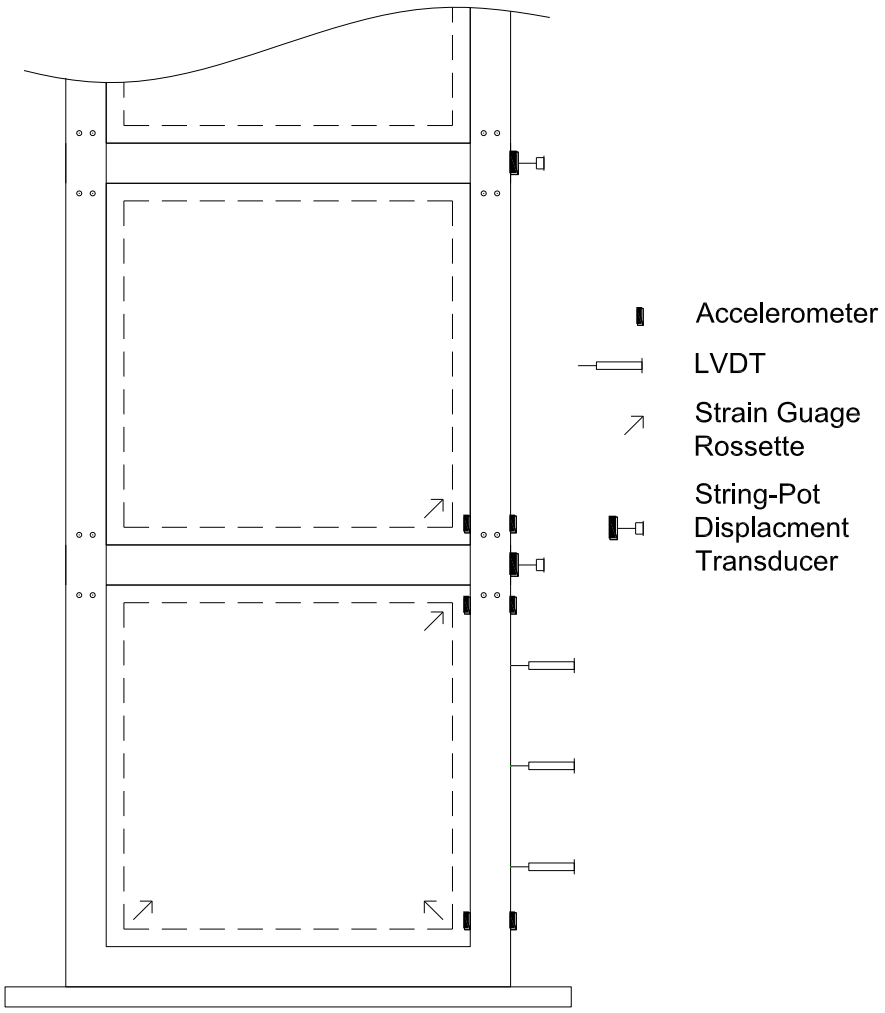


Figure 2-6 Referenced Sensor Layout

Excitation Equipment

The significance of vibration testing in structural dynamics is evident as there are numerous benefits and informative conclusions that can be drawn and applied to structural engineering applications. This section describes shaker systems and their functionality in more detail to provide a baseline of information of shaker categories and components as they pertain to structural dynamic research.

Specifications and Categories

A variety of shakers are available in multiple areas of engineering with different capabilities and principles of operation. Before developing a testing program, the appropriate selection of shaker device must be made. Shakers can be evaluated by three significant performance specifications; force, power, and stroke ratings.

The force rating is the maximum force that could be applied by the shaker to a test object with a mass within the design load of the shaker. Maximum force is usually achieved at higher frequencies in the operating frequency range of the shaker. Similar to force capabilities, the power rating is most useful in the moderate to the high frequency excitations. The stroke rating quantifies the maximum displacement a shaker is capable of moving while exciting an object within the payload limit and is only achieved at very low frequencies. It is important to understand the relationship of these specifications and their limitations. It is not practical to operate a shaker at its maximum stroke and acceleration ratings simultaneously since each of these parameters peak at different frequency ranges. Figure 2-7 shows a performance curve illustrating a function of this relationship and demonstrates the velocity performance limits as a function of stroke, velocity, and acceleration. Plotted in the frequency domain, this curve presents the limiting specifications of a shaker system and can vary depending on the system specifications. Ideally, a performance curve looks like the diagram in Figure 2-7 and has a linear displacement-amplitude region, a constant velocity-amplitude region, and a linear acceleration-amplitude region for low, intermediate, and high frequencies, respectively. Additionally, as the mass increases the acceleration and velocity performance decreases and becomes less able to function at full capacity. It is also

important to realize from this curve that the stroke, velocity, and acceleration ratings cannot be simultaneously achieved.

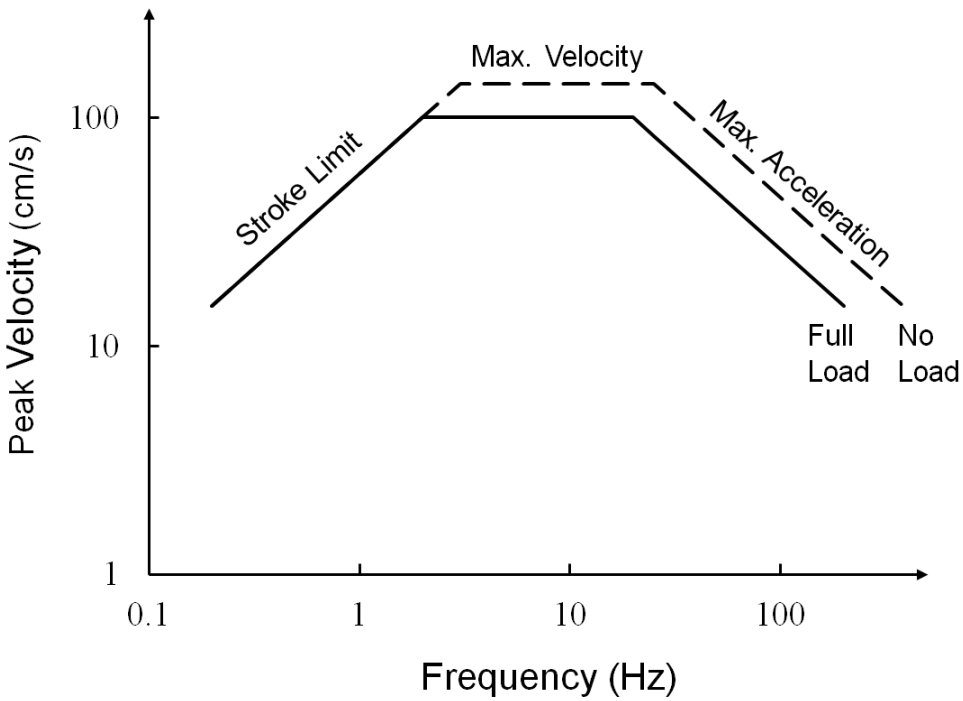


Figure 2-7 Performance Curve for Shaker Systems (DeSilva 2007)

In structural dynamic applications, as well as other engineering disciplines, there are three basic types of shakers widely used: hydraulic, inertial, and electromagnetic systems. In this section the components, processes, and capabilities of each system type are presented.

Hydraulic shakers. Hydraulic shaker systems predominantly depend on the flow of hydraulic fluid to facilitate motion. The main system components consist of a piston-cylinder actuator, servo-valve, fluid pump, and a driving electric motor or power supply. Figure 2-8 identifies the basic components of a typical hydraulic shaker.

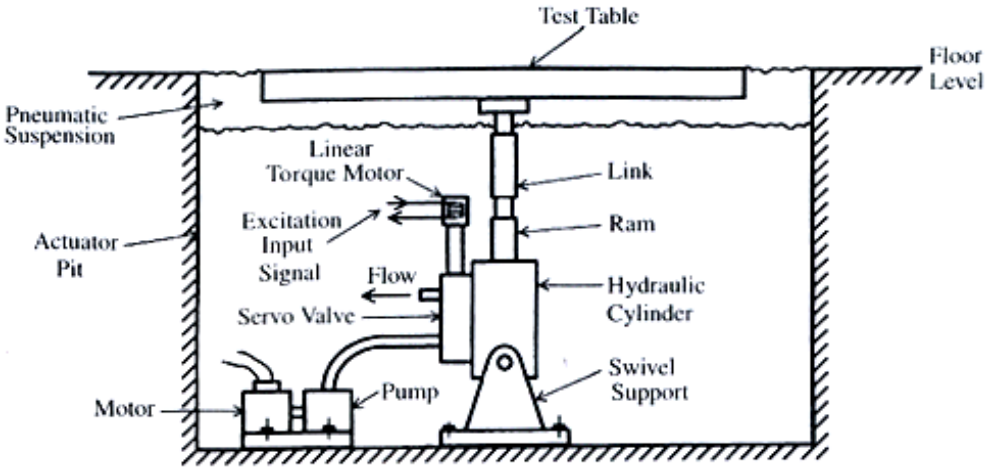


Figure 2-8 Typical Hydraulic Shaker Arrangement (adapted from De Silva, C. (2007). *Vibration, Monitoring, Testing, and Instrumentation.* Boca Raton, Florida: Taylor & Fransis Group)

The general process of a hydraulic system involves the pumping of pressurized hydraulic oil (pressure ~ 4000psi) into the cylindrical actuator through a servo-valve by means of a pump that is driven by an electric motor (power ~ 150HP). The servo-valve regulates the flow rate (~ 100gal/min) of the hydraulic oil entering the actuator which directly controls the resulting piston motion. A typical servo-valve consists of a two-stage spool valve which provides adequate pressure differential and a controlled flow to the actuator, which sets it in motion. Hydraulic shakers provide high operation flexibility during the test and are capable of producing variable-force, constant-force and wide-random-input testing. One drawback to hydraulic systems is the inability to reproduce accurate excitations at high frequency levels due to the presence of distortion that are introduced in this frequency range. These systems can be applied to heavy load testing with larger scale structural models and can include industrial and civil engineering applications due to their dependable low to intermediate frequency operation potential (DeSilva 2007).

Inertial shakers. In inertial shakers, the force that causes the shaker motion is generated by inertial forces or accelerating masses, a fundamental principle that has been integrated in civil engineering dynamic testing since the 1930s. Components consist of two counter rotating masses (rods) and a variable-speed electric motor with a connected gear mechanism. The two equal masses rotate at an identical angular speed and along the same radius of motion but in opposite directions. A resultant sinusoidal force in the direction of symmetry of the two rotating arms is then produced. Each mass has a series of slots where mass can be added and various force magnitudes can be achieved. The masses are driven by an electric motor through a gear mechanism that typically provides several speed ratios that depend on the desired test-frequency range. These shaker types are capable of reproducing intermediate excitation forces which are limited by the strength of the carriage frame in which it is embedded. Excitation motions are exclusively sinusoidal and forces magnitudes are directly proportional to the square of the excitation frequency. Inertial disadvantages include the testing of complex/random excitations and constant force generation along with the inability to vary force amplitude. Despite this limitation, this shaker type is quite effective for sine-dwell and sine-sweep test as it provides a sinusoidal excitation with virtually no distortion and with the addition of a variable-speed motor, frequency and amplitude levels can be preset to incrementally vary during a test (DeSilva 2007).

Electromagnetic shakers. Figure 2-9 is a schematic of an electromagnetic shaker arrangement with followed by supporting context.

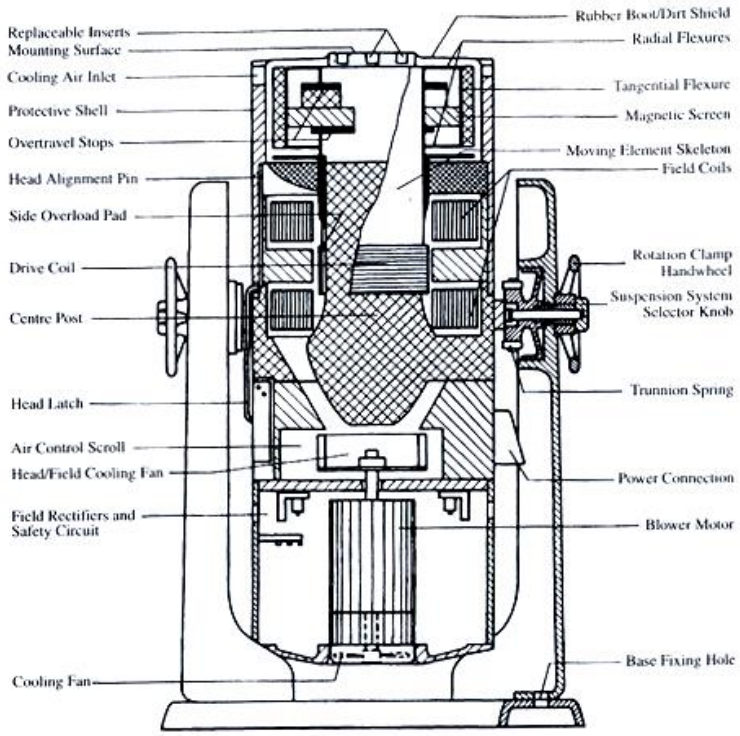


Figure 2-9 Sectional View of Electrodynamic Shaker (adapted from De Silva, C. (2007). *Vibration, Monitoring, Testing, and Instrumentation.* Boca Raton, Florida: Taylor & Fransis Group)

Also called “electrodynamic shakers”, electromagnetic systems rely on the operational capacity of an electric motor and the fundamental forces produced when an electrical excitation signal is passed through a moving coil placed in a magnetic field. There are a variety of complex components included in an electromagnetic shaker system; as a result these shakers are typically prefabricated by a supplier for immediate use and require minimal set up. The theory on which this shaker type is based is briefly explained. A stationary electromagnet consisting of field coils that are wound on a ferromagnetic base generates a steady magnetic field. When the electrical signal is passed through this coil, the shaker head, which is supported on flexure mounts and has its own concentrically wound coil, is set in motion. The benefits of this system include high frequency range of operation, flexibility of operation, and a high level of

accuracy of the generated motion. To properly utilize the advantages of an electromagnetic shaker, applications are typically limited to smaller scale test specimens. Shaker specifications and types are organized in Table 2-4 for a comprehensive review of this section (DeSilva 2007).

Table 2-4 Capability Ranges for Various Shaker Types (De Silva 2007)

Shaker Type	Typical Operational Capabilities				
	Frequency	Max. Disp. (Stroke)	Max.Vel.	Max. Accel.	Max Force
Hydraulic	Low (0.1-500 Hz)	High (20 in; 50 cm)	Intermediate (50 in/sec; 125 cm/sec)	Intermediate (20 g)	High (100,000 lbf; 450,000 N)
Inertial	Intermediate (2-50 Hz)	Low (1 in; 2.5 cm)	Intermediate (50 in/sec; 125 cm/sec)	Intermediate (20 g)	Intermediate (1,000 lbf; 4,500 N)
Electromagnetic	High (2-10,000 Hz)	Low (1 in; 2.5 cm)	Intermediate (50 in/sec; 125 cm/sec)	High (100 g)	Low to intermediate (450 lbf; 2,000 N)

Shake Tables

Shake tables were first used for simulating seismic loads on structures in the 1940s; and after proving to be the most realistic and cost effective avenue to reproduce ground excitation, their use has been widespread since the 1960s (Williams and Blakeborough 2001). The process of utilizing a shake table for dynamic testing is relatively straightforward: a structural model is positioned on a stiff platform or table, which is shaken to apply the appropriate base motion to the model. In seismic studies, excitations can be applied to simulate a particular earthquake record. This testing capability is a major advantage of shake table applications since the building response is a result of base motion as opposed to an attached loading mechanism, thus providing a realistic portrayal of the structure's loading and response. Assuming ground excitation

can be accurately applied, inertial forces are then generated throughout the structure and the system response can be monitored.

The main components of a shake table system consist of a table or platform to mount a test specimen and the connecting actuators responsible for its excitation. The table must be adequately rigid so that it does not resonate during a test and it transmits the true input motion to the structure with minimal deviation from extraneous energy dissipation.

A variety of table platform designs have been implemented. UC San Diego has a 2.2 m shake table consisting of a steel plate surrounded by a torsional stiff shell and internal stiffening honeycomb (Van de Einde 2004). An alternate table design can be found at UC Berkeley where a shake table platform consists of a ribbed, post tensioned concrete slab (Blakeborough et al. 1986).

In order to drive the large mass of the shake table coupled with the test specimen at the required rate, high-capacity servo-hydraulic equipment is typically implemented. In such a hydraulic system, the velocity content of the earthquake records are usually the governing factor of a simulation since this is directly related to the oil flow rate regulated by the pumping system and servo valves. For standardized table systems hydraulic actuators and the associated servo valves must fall well within the range of current technological capabilities.

In 1999, the National Science Foundation (NSF) launched the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES), a program to develop experimental and computational facilities for earthquake engineering research. By establishing this research conglomerate, earthquake-engineering studies became a

collaborative effort among the academic community rather than a collection of individual projects. The NEES network features 14 laboratories distributed across the nation and as listed in Table 2-5.

Table 2-5 NEES Facilities by Region

West Coast	Central	East Coast
University of California, Santa Barbara	University of Texas, Austin	University at Buffalo, SUNY
University of California, Davis	University of Minnesota	Cornell University
University of California, Los Angeles	University of Illinois, Urbana- Champaign	Lehigh University
University of California, San Diego	-	Rensselaer Polytechnic Institute
University of California, Berkeley	-	-
University of Nevada, Reno	-	-
Oregon State University	-	-

With this collective group of researchers, experimental capacities have not only grown to include large-scale shake tables, but tsunami wave basins, geotechnical centrifuges, and field experiments/monitoring as well (NEES 2007).

Shake tables systems come in a wide variety of sizes and configurations. In the United States the largest shake table systems are at a few select locations; the University of California at Berkeley (20ft x 20ft, 6DOF), the State University of New York at Buffalo (12ft x 12 ft, 5DOF), and the University of Illinois at Urbana-Champaign (12ft x 12ft, 1DOF). A review of these and other U.S. shake table testing facilities is given in Table 2-6 (Nigbor and Kallinikidou 2004).

Table 2-6 Largest Shake Table Facilities in the U.S

Institution	State	Payload (metric ton)	Size (m×m)	DOF	Freq. Range (Hz)	Max Stroke (m)	Max Velocity (m/s)
University of California San Diego	California	2000	7.6×12.2	4	0-33	0.75	1.8
EERC, University of California Berkeley	California	45.36	6.1×6.1	6	0-15	0.127	0.762
State University of New York at Buffalo	New York	50	3.6×3.6	5	0.1-50	0.15	1.25
University of Nevada at Reno	Nevada	45	4.3×4.5	2	0.1-30	0.3	1
U.S. Army, Civil Engineering Research Lab, Illinois	Illinois	45.36	3.6×3.6	3	0.1-60	0.3	1.3
Wyle	Alabama	27	6.1×6.1	2	0-100	0.152	0.89
University of Illinois at Urbana-Champaign	Illinois	4.5	3.7×3.7	1	0.1-50	0.05	0.381

Unique to other shake table facilities, the University California at San Diego recently constructed the first outdoor and largest shake table facility in the U.S. Measuring 25ft x 40ft with a vertical payload of 20 MN, this Large High Performance Outdoor Shake Table (LHPOST) significantly improves the capabilities of the NEES research program. Previously identified limitations of existing shake table systems are payload, hydraulic power supply, and stroke but also include overhead room to construct and test a tall structural system. The LHPOST is designed to nullify the vertical space constraints and significantly increase payload capacities providing a new avenue for full-scale model testing (Van Den Einde et al. 2004).

Despite the local significance and payload capacities, domestic shake tables identified in Table 2-6 are still considered medium sized tables compared to those overseas, with the exception of the recently constructed LHPOST. Typically, specimen size limitations for smaller sized shake tables can be viewed as a disadvantage due to problems associated with scaling of nonlinear dynamic responses. The exception is in Japan where some very large and expensive facilities have been constructed and dynamic testing can be conducted on full-scale structures. In particular is the E-Defense shake table, which has three translational and three rotational degrees of freedom in the X-, Y-, and Z-axis and measures 20m x 15m in size. With a total of 24 actuators (five units in the X- and Y- axis's and 14 in the Z-axis), this table system is the largest and most complex system in the world. Recent experiments conducted include a long period seismic response of a large-scale high rise building (Chung et al. 2010) and earthquake effects on a full-scale, six story, light-frame wood building (van de Lindt 2010). Shown in Table 2-7 are some of the shake table facilities located in Japan, identifying their size and capacities.

Table 2-7 Major Japan Shake Table Facilities

Institution	Payload (ton)	Size (m×m)	DOF	Freq. Range (Hz)	Max Stroke (m)	Max Velocity (m/s)
NIED E-Defense	1200	20×15	6	0-50	1	2
National Research Institute for Earth Science and Disaster Prevention	1088	20×15	3	0-15	1	2
Nuclear Power Engineering Corp	1000	15×15	1 horz& vert	0-30	0.2	0.75
NRC for Disaster Prevention, Tsukuba	500	15×15	1 horz& vert	0-50	0.03	0.37
Public Works Research Institute	272.11	8×8	1 horz& vert	0-50	0.6	2

Although electromagnetic shakers have established their value in selected engineering applications (Reynolds and Pavic 2000) and inertial shakers gave rise to structural dynamic testing in the early 1930s, hydraulic systems are becoming the most prevalent for shake table systems in structural engineering research. Specific actuator placement, overall motion versatility, and collectively large payload capacity has made this system type rise above others as the most practical and effective method for shake table testing. It is evident by comparing shake table capabilities in the U.S (Table 2-6) and Japan (Table 2-7) that large to full-shake table testing is a high priority overseas. Despite this contrast, collaborative research can continue to be done with existing facilities to strengthen the structural engineering field.

CHAPTER 3
EXCITATION EQUIPMENT: MANAGEMENT AND IMPLEMENTATION

The selection and implementation of appropriate excitation equipment is necessary for their essential ability to input energy into a structural model or system. Chapter 2 highlighted the capabilities and performance criteria for three types of excitation equipment, two of which are adopted in the dynamic testing laboratory presented in this work: hydraulic and electro-magnetic systems. This chapter will identify three different excitation instruments incorporated into the testing facilities: (1) a uniaxial shake table, (2) a long stroke shaker, and (3) a six degree-of-freedom shake table.

Small Scale Equipment

With the broad application of dynamic testing and research in both industry and academia, excitation equipment packages are widely marketed and made available for a number of test applications. These packages, which often provide the necessary hardware and control software, provide an expedited approach to populating a laboratory with testing equipment that may be easily assembled and rapidly integrated into a testing program. This section will identify two small-scale excitation systems adopted in this manner: a long stroke shaker and a uniaxial shake table. Performance specifications, research applications, and an overall review of each piece of equipment will be presented to illustrate how the integration of such systems assists in creating a versatile structural dynamics research laboratory.

Long Stroke Shaker. An APS 113 electrodynamic long stroke shaker has been incorporated into the laboratory facility for a range of test applications. The shaker operates in conjunction with a power amplifier that provides the motion signal to the

shaker. When used with an appropriate function generator (such as LabView and an NI analog output module), this versatile system can be used to generate an array of motion types including a general sine wave or sine sweep variation, random white noise, or impulse loadings. The “long stroke” feature allows for large displacements at low frequency vibrations. Figure 3-1 shows the long stroke shaker installed on the ground floor of the laboratory followed by Table 3-1 which presents some highlighted performance specifications for the long stroke shaker system

Figure 3-1 Long Stroke Shaker

Table 3-1 Highlighted Long Stroke Shaker Specifications

Long Stroke Shaker Specifications	
DOF	1
Force (lbf)	30
Stroke (in)	6.25
Velocity (in/sec)	39
Freq. Range (Hz)	0 – 200
Operation	Horizontal or vertical

For the tests discussed in detail in Chapter 4, this system is configured to apply a band limited white noise excitation to a structural bridge model for experimental modal analysis. Bolted to the floor of the lab, the shaker has a fixed base and is attached to

an isolated bottom node of the bridge (Figure 3-1). Using the signal amplifier, generated signals can be adjusted to the appropriate magnitude for the particular testing application. With the unique ability to operate in both the horizontal and vertical modes along with the long stroke capacity, this instrument has the potential to be a useful tool in the laboratory for current and future research interests.

Uniaxial Shake Table. In contrast to the long stroke shaker, which is configured to impart a single point vertical excitation force to a structural model, the laboratory also includes a bench-scale uniaxial Quanser shake table, suitable for providing horizontal base excitation to structural models. Isolated on a rigid concrete table, this mechanical shake table has the ability to impose excitation to a 17-pound mass at a maximum acceleration of 2.5g's in one translational direction. The Quanser shake tables are used by researchers and educators worldwide for structural dynamics demonstrations and small scale testing. Displayed in Figure 3-2 is the uniaxial shake table as it is installed in the lab and Table 3-2 highlights additional performance specifications of the shake table system.

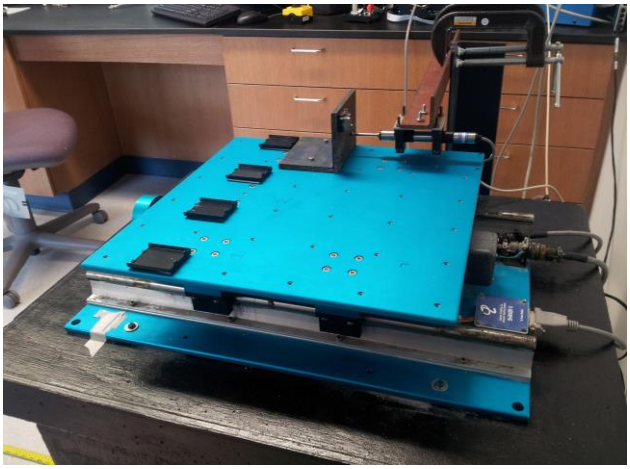


Figure 3-2 Uniaxial Shake Table

Table 3-2 Highlighted Uniaxial Shake Table Specifications

Uniaxial Shake Table Specifications	
DOF	1
Table Size (ft)	1.5 x 1.5
Payload (lb)	17.1
Force (lb)	159.3
Stroke (in)	3
Velocity (in/sec)	26.18
Acceleration (g)	2.5

The control system of the shake table utilizes an accelerometer to measure the generated shake table motion to provide the feedback for the control loop. Figure 3-2 illustrates the use of an external LVDT sensor to measure reference displacement data of any imposed vibration during a test. This system has been applied to numerous types of structural dynamic research applications including both random vibration and harmonic motion testing of a small scale structural building model which enables an array of post processing system identification methods to be implemented.

Both excitation devices presented in this section are small scale models which have been implemented in the lab for early stage sensor development for structural health monitoring research and can be used further for structural dynamic experiments. The specifications of each are clearly listed with critical performance limits identified to highlight the importance of understanding the capabilities of a system. In both cases this excitation equipment required minimal assembly and allowed for prompt application to research experiments after a thorough review through the users manuals of each. When a new piece of excitation equipment is included in a lab, it is good practice to

develop a testing protocol or customized user's manual to enable its safe and effective use. Following the development of the appropriate guides, the combination of these two compact pieces of excitation equipment have provided the tools needed for a versatile structural dynamic research laboratory.

Large Scale Equipment

The required size and complexity of excitation equipment must match the models and components to be tested; larger experiments require larger testing equipment. To expand the dynamic testing capabilities of the laboratory identified in this work, a larger scale excitation system was implemented. This section will provide a detailed outline of all project phases from the system selection through the installation process. Additional information on system performance and experimental applications will also be included to emphasize the significance of incorporating such a system to a testing program.

Shake Table. A six degree-of-freedom (6DOF) shake table was the largest and most complex addition to the laboratory facilities developed in this research. Unlike the previous excitation devices presented in this chapter, this excitation system did not come ready to use and required detailed assembly and installation to be properly calibrated and experimentally functional. Intended to be housed in the University of Florida Powell Structures Laboratory, this shake table provides seismic research capabilities in addition to the existing tornado, hurricane, and blast experimental capabilities at the lab. During the design phases of an expansion to Powell Lab, it was decided that the new shake table system would be purchased and included in this newly constructed building.

Purchased from a servo-hydraulic test equipment manufacturer (Shore Western Manufacturing, Inc.), this predesigned system has main components consisting of a 4' x

4’ aluminum shake table platform on which to attach experimental models, six hydraulic actuators used to drive the table into motion, and a hydraulic service manifold used to regulate hydraulic fluid values from low to high pressure operations. Identified in Table 3-3 are highlighted shake table specifications.

Table 3-3 Highlighted Shake Table Specifications

Large Shake Table Specifications		
DOF		6
Table Size (ft)		4 x 4
Payload (lb)		2,200
Stroke (in)	Z	3 in
	X & Y	6 in
Velocity (in/sec)		20 (X,Y,Z)
Acceleration (g)	Z	2.0 g
	X & Y	4.0 g
Freq. Range (Hz)		0 - 200

One unique feature of this shake table system is the ability to apply both translational and rotational motion in the X-, Y-, and Z-axis, resulting in the six available degrees of freedom identified in Table 3-3. To accommodate potential seismic accelerations up to 4g’s, a large mass foundation was required. The purpose of this foundation, typically designed with a mass 50 to 100 times the table and payload mass, is to approximate a rigid support condition and ensure that vibration is not transferred to the surrounding structure. Figure 3-3 shows the initial and completed construction phase of the isolated mass concrete foundation for which the newly integrated shake table system would be installed.

A.  B. 

Figure 3-3 Foundation Construction

This heavily reinforced concrete foundation was dimensioned and designed to withstand increased lateral forces in addition to the vertically applied forces coupled with the overall weight of the system. The inside perimeter of the pit is a L6”x6”x3/8” continuous steel angle fortified by three L8”x8”x1 1/8” intermediate sections inserted in areas where the horizontal actuators will be bolted. Displayed in Figure 3-4 is an as-built schematic of the shake table pit a week after construction.

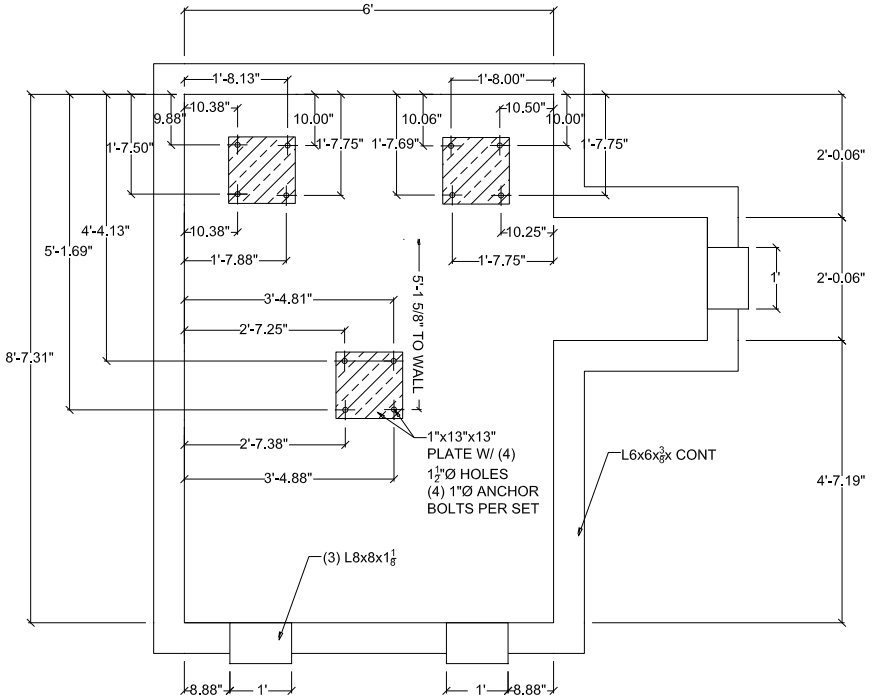


Figure 3-4 Shake Table Foundation As-Built

Protruding out of the concrete surface at the bottom of the pit, were three sets of (4) 1” diameter threaded steel rods coupled with a 1” thick steel plate to fix the three vertically oriented actuators. Each plate rests on (4) 1” anchor bolts to allow for the bottom of each actuator to be equally leveled and is fastened by an additional (4) anchors bolts above to resist any uplift forces during full scale operation.

Following the delivery of the shake table system from the manufacturer, the assembly was initiated. The unique shape of hydraulic actuators coupled with their 200-lbself-weight made their transportation from the packaged crate to the concrete pit an installation challenge. Careful project planning during this installation process was essential to effectively install this system while maintaining a superior level of safety. Figure 3-5 shows the actuator shape and the gantry crane used to lift and transport the set of actuators and the shake table platform.

A.  B. 

Figure 3-5 Shake Table Assembly

Shoring was designed and constructed to place the horizontal and vertical actuators in the appropriate position and were collectively used as temporary support for the shake table platform. Actuators were then bolted to a torque of 150lb-in to predrilled holes on the shake table perimeter and leveled to ensure installation accuracy. This torque spec

was then applied to all actuator bolted connection to the surrounding supports. After all major components of the system were properly connected and leveled, hydraulic hoses were connected from the hydraulic manifold system to each actuator according to the specified hose connection layout provided by the manufacturer. To generate hydraulic fluid pressure, the shake table system is powered by 125 HP hydraulic power supply with a maximum pressure output of 3000psi. This power supply system was designed and manufactured by an external company to meet the specific operational requirements of the shake table. As a final assembly step, the temporary shoring beneath the table was removed to allow for the system calibration phase of the project to begin. Shown below in Figure 3-6 is the final assembly of the shake table system as it is currently in the lab.

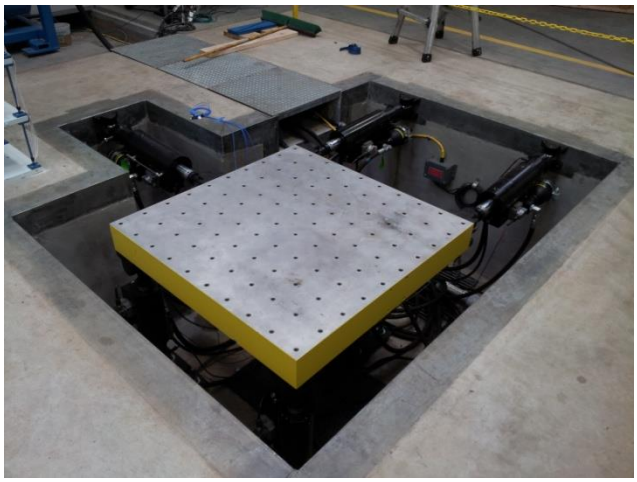


Figure 3-6 Installed Shake Table System

In addition to the hardware components of the system, a central processing unit with a preloaded control interface was delivered to function as the main control system of the table. The control system enables multiple variations of motion to be manipulated and with the integration of an LVDT and accelerometer sensor at each actuator, both displacement and acceleration controlled excitation can be applied.

The shake table manufacturer provided personal technical support for one week. This support enabled an initial professional calibration of the shake table system as well as thorough education on system performance and control program interface. Following the on-site commissioning, a Shake Table Instruction Manual (Appendix 1) was composed. This document provides a detailed description on the multiple phases of operation pertaining to the intended structural dynamic research experiments of the facility.

With the multiple excitation degrees of freedom provided by this machine, there is a wide range of experimental application possibilities. Upcoming experimental tests utilizing the shake table include random white noise base excitation on a steel building model to characterize structural dynamic properties, as discussed in Chapter 4. Additional forced vibration can be applied by this system such as uni- and multi-axis seismic acceleration time histories or harmonic vibration variations. It must be noted that there is an elaborate convergence process associated with any new excitation record applied to the shake table system in order to align the desired input motion with the resulting response of the table. Once an input signal and output motion of the table is converged, this motion can be stored and immediately applied to experiments in future applications. This summarized convergence process is detailed in the Shake Table User's Manual included in the Appendix C section of this document. An effective method to ensure the appropriate alignment of an input signal and output response is the plotting of a transfer function between each. Plotting this relationship provides a frequency domain visual to determine if the signals are equal in content. For example if two signals are truly identical with one another, the transfer function ratio plot should be

a constant line with a magnitude of 1. Figure 3-7 presents a time history displacement record of two representative white noise signals with frequency content ranging from 0-2.5 Hz. Following is a transfer function plot between the two sources in Figure 3-8.

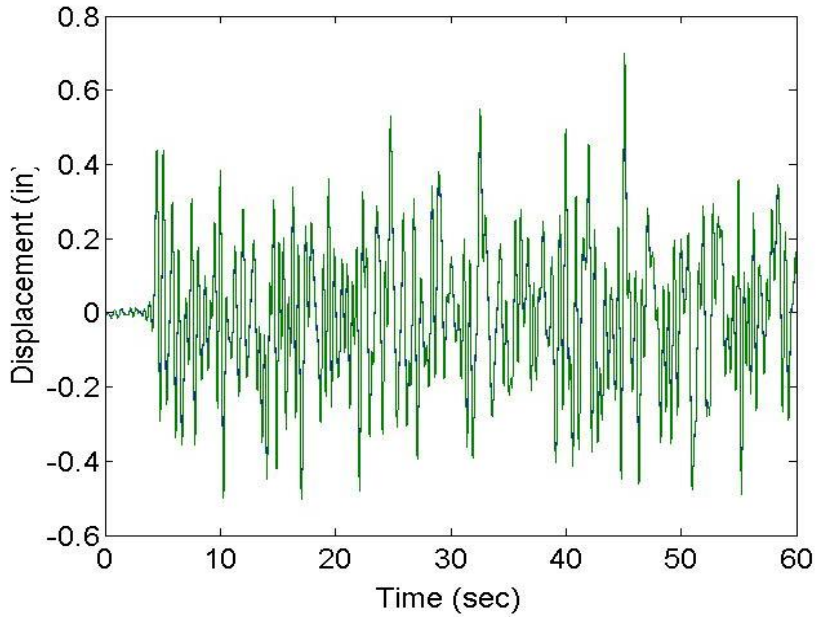


Figure 3-7 Displacement Time History of Input and Output

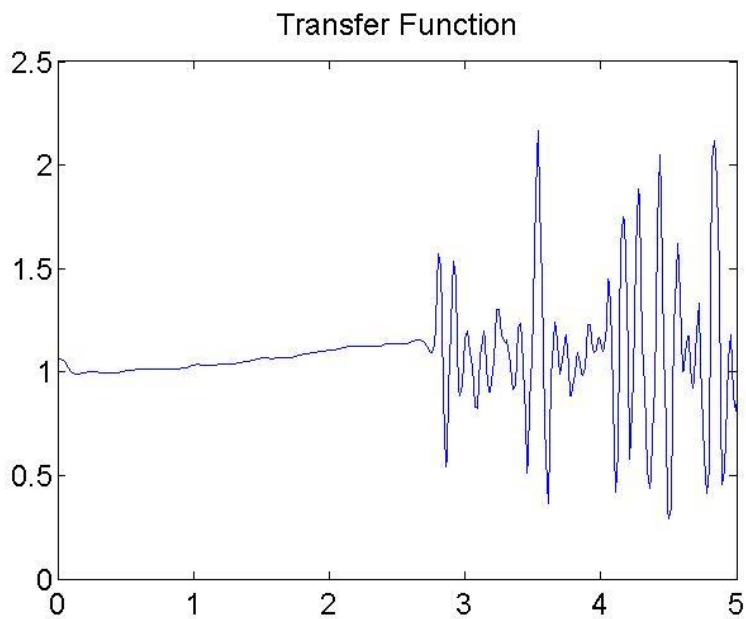


Figure 3-8 Transfer Function of Input and Output

In the time domain, it can be difficult to observe the subtle differences when comparing two signals. Plotting the transfer function provides a visual ratio in the frequency domain that can be a more effective platform to observe signal inconsistencies. The visual presented in Figure 3-8 illustrates a reasonably successful convergence as indicated by the relatively constant graph from the 0-2.5 Hz frequency range of the input and output signals.

As a future recommendation, a system performance characterization program should be developed to quantify the true limit states of the system. Figure 3-9 plots the intended performance curve of the shake table system under a fully loaded condition (added 1 ton mass) according to the system design and power supply specifications.

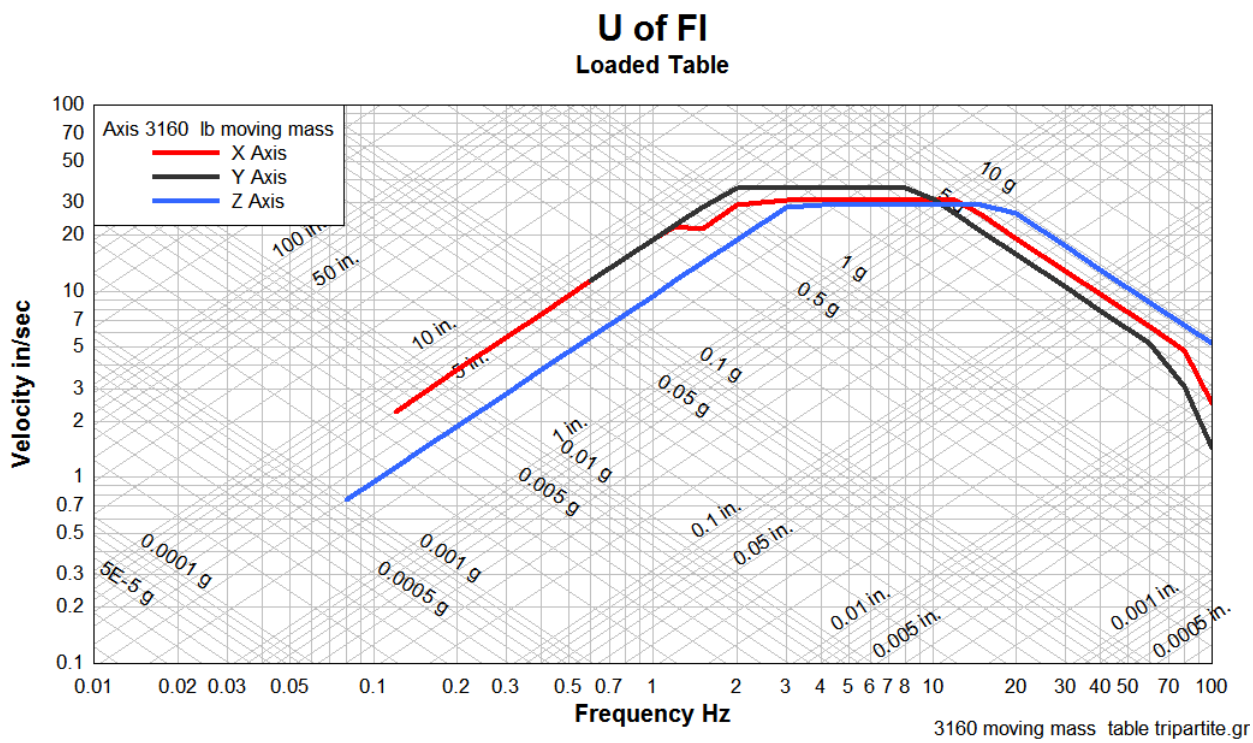


Figure 3-9 Intended Performance Limits (Shore Western Manufacturing, Inc.)

The stroke, velocity, and acceleration specifications identified earlier in this section in Table 3-3 originate from this performance curve which was created by a computer

model (Shore Western Manufacturing, Inc.) to predict system capabilities. To provide a contextual interpretation of the information presented in Figure 3-9, a brief explanation of each limit referring to the X-axis will be presented here. A translational stroke limit of six inches can only be obtained at a low frequency range from 0–2Hz. After which, in the intermediate frequency range of 2-12Hz, a maximum velocity limit of 30 in/sec is the controlling factor of motion. Frequencies higher than approximately 12Hz can be achieved but are controlled by a maximum acceleration limit of 6g's. Of these three controlling factors, a six inch stroke, is the only limit conserved when comparing theoretical capabilities based on system design (Figure 3-9) to the specified limits of the system (Table 3-3). As a conservative measure, the specified X-axis velocity and acceleration limits of 20in/sec and 4.0g's are reduced from the idealized system capabilities from Figure 3-9 above. With unique variability in any hydraulic system, the value of these parameters may change. As a result, it is important to incorporate a performance characterization test program for the shake table unit to compose a SIMLab specific performance curve and ensure true performance meets the specified limits.

A uniaxial shake table, long stroke shaker, and 6DOF shake table have all been identified as valuable pieces excitation equipment integrated into the research facility presented in this work. With the proper operational instruction and background knowledge of these systems, there exists a matrix of experimental possibilities for which these machines can be applied.

CHAPTER 4
STRUCTURAL MODELS: DESIGN AND TESTING

A key aspect of the development of the structural dynamics laboratory undertaken in this research is the creation of a number of structural models with a range of design, construction, and dynamic performance characteristics. This chapter presents three different structural models designed and implemented for dynamic testing, including a steel truss bridge, a small-scale two-dimensional building, and a three-dimensional steel building model. The design, assembly/construction, and experimental applications of each model are identified followed by further discussion on improvements and future experiments.

Bridge Truss

In this section, a laboratory-scale, steel truss bridge is presented for use in dynamic experiments. The bridge, used in previous academic applications, was originally designed and built for structural health monitoring research and damage identification applications at Texas Tech University. The truss was modified as part of the work presented in this thesis in an effort to improve its applicability for a range of dynamic tests.

A detailed and comprehensive description of the original truss bridge design is presented in Hernandez (2011); however, a general overview is briefly provided here. The bridge model was designed as a 1/6th scale model of an existing steel bridge located in the North West Texas panhandle, maintaining general material and boundary conditions of the original bridge. The model size and design is a reflection of geometric similitude relationships aimed at closely matching the dynamic characteristics of the model with those measured on the real bridge. A subsequent iterative design

procedure, coupled with finite element model predictions of dynamic characteristics, was the basis for member cross-sectional dimensions.

In addition to achieving the desired dynamic characteristics, the bridge was also designed with specific functionality to make it useful in testing damage detection algorithms. As such, the bridge was designed to be able to impose “damage” by interchanging original truss elements with elements possessing smaller cross-sectional dimensions. To achieve this element interchangeability, specially designed connection blocks were fabricated, as discussed later in detail.

The resulting model was a simply supported, steel bridge truss spanning 10ft with rigid connections at each nodal location, as shown in Figure 4-1.

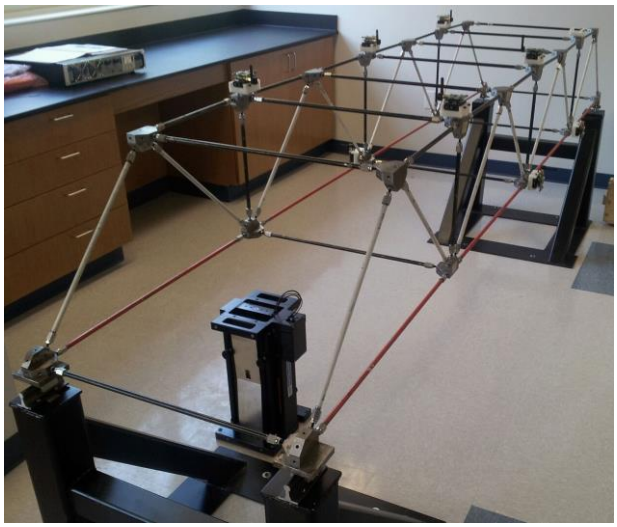


Figure 4-1 Erected Bridge Model

The bridge had a width measuring 2ft and had a top and bottom deck separated by a height of 1.25ft. Observed in Figure 4-1, the bottom deck is divided into four 2ft x 2.5ft rectangular bays. Because this structure was designed, fabricated and originally constructed at another institution, the model’s design and resulting dynamic properties were inherited by the dynamic research facility presented in this work. Therefore, this

structure provided a rapid and prefabricated approach to begin to populate the facility with new structural dynamic models.

The objective of this portion of the bridge work is to present the systematic assembly and experimental testing of the bridge and to quantify dynamic characteristics. This process illustrates key issues in practical dynamic testing on structural models in addition to providing known fundamental parameters of the bridge model for future use. The testing techniques, including instrumentation methods and data processing procedures, are discussed to provide a foundation for future experiments on structural models.

Assembly and Modifications

The assembly of a structural model used in dynamic testing requires a level of consistency and precision to ensure the accuracy and legitimacy of experimental results and conclusions. This section outlines a general assembly process and identifies the equipment used to ensure the integrity of the fully-assembled model.

The disassembled bridge model was shipped to UF in groups of like components. Figure 4-2 shows the various bridge components, consisting of steel truss members, screw/bolt fasteners, and nodal connection blocks.

Figure 4-2 Bridge Components

With the erector set-like composition of the model, precise construction methods were required to preserve dimensional accuracy and overall symmetry of the system dictated in the design drawings.

Each nodal block is designed to connect adjacent steel members in to a semi-rigid connection which provides a continuous chord (two for both top and bottom levels) of hollow steel members spanning the length between supports. This is made possible by the combination of steel screws, bolts, and nuts, connecting the connection block to the steel truss member. An example of a member-to-connection block assembly is shown in Figure 4-3.

Figure 4-3 Joint Configuration

This member connection is applied throughout the entire structure. The bottom level bridge deck was erected first, followed by the connection of individual vertical truss members to each node. The top deck level was then used as the final component to unite the system components. Accuracy to the thousandth of an inch was maintained with the utilization of a large 32-inch caliper to verify that the center-to-center dimensions of each truss element in the physical model were consistent with the design specifications.

While all of the components of the original truss were sent from Texas Tech, the truss supports had to be fabricated at UF to elevate the truss off the ground. The truss support design required the supports to have a high stiffness relative to the truss to ensure that any vibration imparted results in primarily truss vibration and not motion in the supports. The construction of the rigid steel supports shown in Figure 4-1, used a combination of a horizontal band saw and MIG welder. After model erection, to confirm each supported end of the model shared the same elevation, a Dewalt rotary laser level was used as a reference for each end and appropriate adjustments were made to match the heights of each support end. This 'elevation matching' measure ensured equal mass distribution throughout the model and aimed to minimize discrepancy in dynamic response.

In previous work (Hernandez 2011), dynamic experiments identified high vertical stiffness of the model in comparison to the full-scale structure on which this model is based. This discrepancy is likely due to the mass from the wooden decking and cross members of the full-scale bridge that were not accounted for in the finite element model. The result was a decreased mass to stiffness ratio and subsequent lower frequencies (Eq. 2-1) in the modeled full-scale structure. At UF, in an effort to reduce the vertical stiffness of the model, the bridge was extended one extra bay length of 2.5ft to create a total span of 12.5ft. The goal of lengthening the bridge was to decrease its natural frequencies of the vertical direction and provide a larger bridge model with increased nodal variety to be exposed to dynamic experiments. A visual comparison of the bridge extension is illustrated in Figure 4-4.

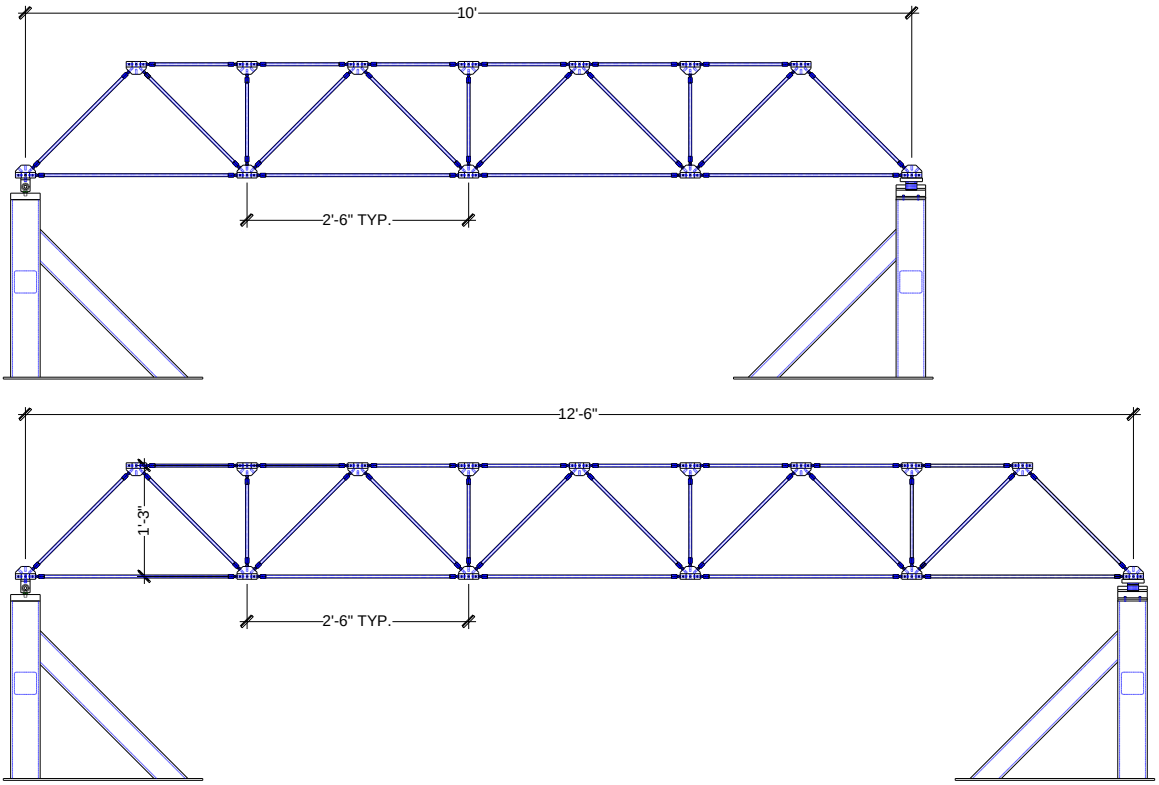


Figure 4-4 Bridge Extension

The results of dynamic experiments and the modification effects of the extension are discussed in the “Experiments and Results” section of this chapter.

One effective characteristic that can be incorporated in the design of a structural dynamic model is the level of experimental adaptability; an extreme case is identified in Chapter 2 (Wu & Samali 2002) where the mass, boundary conditions, and story height can be manipulated to model a range of structural behavior. To apply a similar approach to the model bridge, lateral cross bracing on the bottom deck was installed. This increased the model versatility to not only include an extension, but to also incorporate a lateral bracing dimension to study the dynamic effects of different bridge augmentations. Due to 45 degree face angle of the nodal connection blocks combined with the rectangular shape of the bottom level bays (2.5ft x 2ft), this design modification

required an adjustment to the connection blocks the lateral cross bracing was to be connected to. Figure 4-5 shows a plan view section cut of the bridge and helps clarify the geometric limits of these components and the need for connection block alteration.

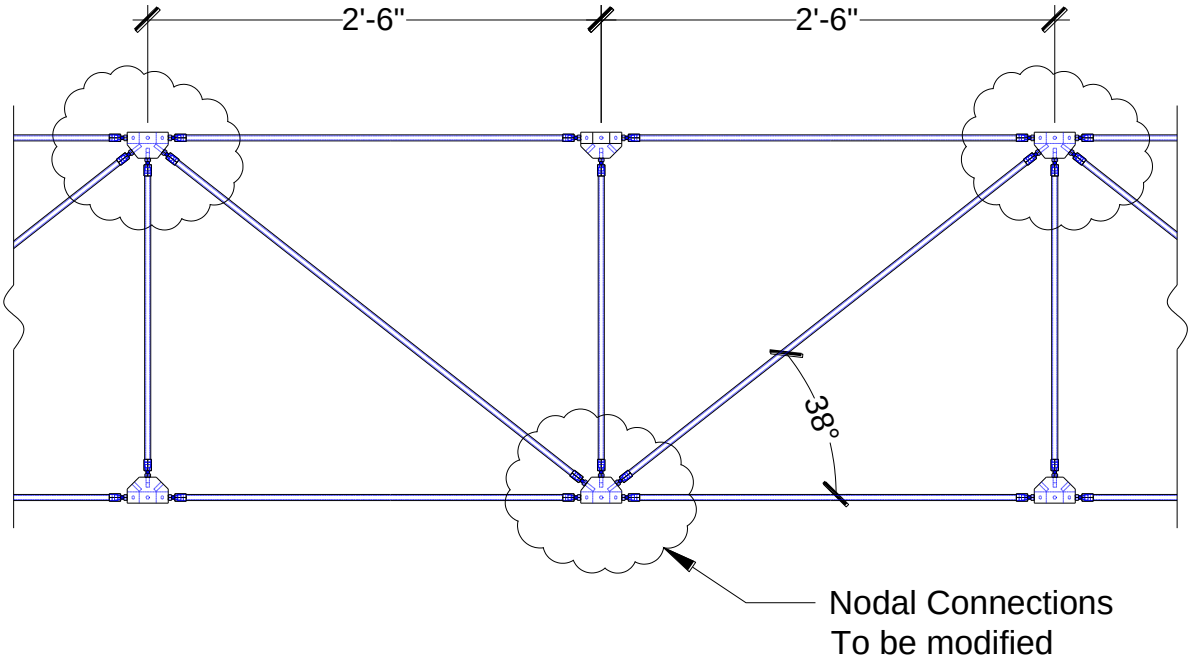


Figure 4-5 Plan section of the bottom deck of the truss to identify the Modified Connection Blocks

After the connection block modifications were complete, a total of three phases of the bridge model were tested; these models include a shorter span without lateral bracing, shorter span with lateral bracing, and longer span with lateral bracing. The dynamic experiments of each bridge configuration are conducted and described later in the section. In appropriate instances, models in this section may also be referred to as (1) for short without lateral bracing, (2) for short with lateral bracing, and (3) for long with lateral bracing.

Finite Element Model

Finite element (FE) modeling of a structural dynamic model enables the prediction of its dynamic response and strength capacity and is a critical stage in the design process. Upon fabrication of the final model design, the anticipated response values can be compared to data obtained through experiments and appropriate adjustments to either the computer model or the test specimen can be made. A general finite element software package (ADINA 8.8) was used to create a finite element model for each configuration of the bridge to be assessed.

All of the bridge FE models, similar to experimental models, contained the same material properties, element cross section properties, and boundary conditions, with the only changes between the configurations being the span dimension and the addition of lateral diagonal members between nodes. To accurately model the existing experimental bridge parameters, the dimensions, nodal locations, and mass were all preserved when transitioning from the real to computer model.

Due to the three-dimensional configuration of the bridge, all six degrees of freedom were left active for the entire model and support conditions were modeled to represent a traditional simple support, with fixity resembling pinned and roller ends. To model these particular boundary conditions the pin nodes were restrained from translation in the X, Y and Z direction and the roller nodes were restrained from translation in only the Y and Z direction.

Beam-pipe elements were implemented to represent steel truss members rather than truss element to allow for all degrees of freedom to be active and both horizontal and vertical natural frequencies could be quantified simultaneously.

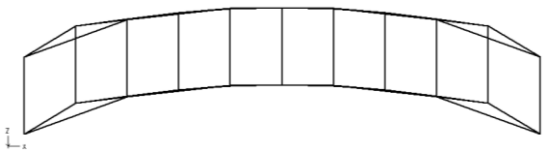
The material makeup of the structure contains three types of components all made up of the same steel material. Material properties and dimensions were consistent with the real model and are listed below:

- E = 30,000ksi
- Pipe area = 0.07 in^2
- Mass Density = 0.00029k/in^3/g
- Span 1 = 10ft
- Span 2 = 12.5ft
- Width = 2ft
- Height = 1.25ft

The natural frequencies calculated from experimental results provide the primary basis for comparing the model with the FE analysis results. The mass of the bridge is an essential parameter for the accurate modeling of a structure for dynamic comparison. The density listed above is only applied to the pipe beam members so only the mass of those elements are accounted for in the FE model. Contained in the lab model are steel connection blocks located at each node that also contribute to the overall mass of the structure. To account for this collective 35 pounds of dead load, a representative point mass of 1.5 pounds was added to each nodal location in the FE model.

Experimental modal analysis is not influenced by loading conditions, with the results being a function of the structure's material, geometric and boundary condition parameters. As a result, element discretization capabilities typically applied to a finite element model around boundary constraints or load concentrations was not required. Figure 4-6 illustrates the first and second horizontal bending modes of the bridge which. Following Figure 4-6 is Figure 4-7 which also identifies the first and second mode but in the vertical direction. These renderings are a result of a finite element

Eigen analysis providing a visual interpretation of the global mode shapes of the structure.

A. 

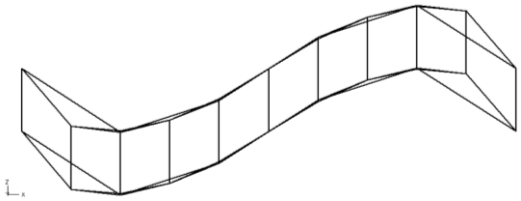
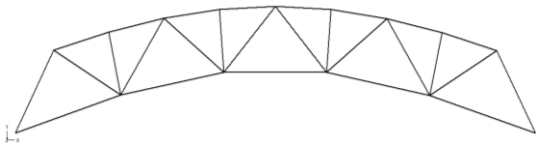
B. 

Figure 4-6 1st & 2nd Horizontal Mode shapes

A. 

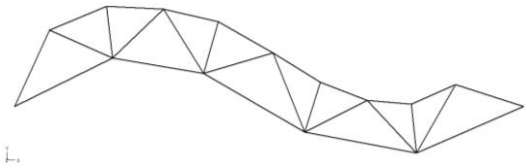
B. 

Figure 4-7 1st & 2nd Vertical Mode shapes

This analysis was applied to all model configurations to provide a general dynamic response estimate of each. Table 4-1 lists FE model predictions for the first three horizontal and vertical modal frequency values for the models (1), (2), and (3).

Table 4-1 FEM Predicted Natural Frequencies

Mode Shape	Frequency (Hz)		
	Original (1)	Short Braced (2)	Long Braced (3)
Horizontal			
1st	5.9	93.76	90.70
2nd	13.0	197.9	194.3
3rd	20.7	204.8	200.1
Vertical			
1st	60.1	60.9	41.1
2nd	107.0	93.5	92.3
3rd	204.0	202.4	196.7

The first three natural frequencies in each direction for each bridge configuration presented in Table 4-1 were determined by plotting the mode shapes in the finite element program and recording the corresponding frequency of vibration. Evaluating these FEM results, it can be seen that the horizontal bracing included in model (2) and (3) adds significant stiffness in the horizontal plane resulting in an increase in horizontal natural frequencies. The functional result of the bracing on the bottom plane is representative of an extremely stiff bridge deck coupled with a moderately flexible beam system above, which more closely resembles a full-scale bridge. By implementing an FE bridge model, dynamic response predictions can be recorded, used as a reference, and may assist in the interpretation of experimental response data in the future.

Dynamic Testing

To obtain fundamental dynamic characteristics for each of the model configurations, a series of structural dynamic tests were conducted. Structural dynamic testing consists of an experimental phase, where data is acquired from a vibration test, followed by an analysis phase, where the data is processed to extract modal parameters such as natural frequencies, modal damping ratios and mode shapes of the structure of interest. This section details the experimental setup, providing detail on the applied excitation methods as well as the instrumentation devices used.

Instrumentation. To monitor the bridge's response during vibration tests, the models were instrumented with high sensitivity uniaxial accelerometers (PCB Piezotronic ICP) specifically designed for modal analysis and structural testing applications. These sensors were used on all structural models presented in this work. The data acquisition system (National Instruments CompactDAQ) to which the accelerometers were connected, provides 20 channels of 24-bit resolution acceleration

measurement. Each of the accelerometers was connected at selected nodes depending on the length of the bridge. Sensor placement on the original and extended models is shown in Figure 4-8.

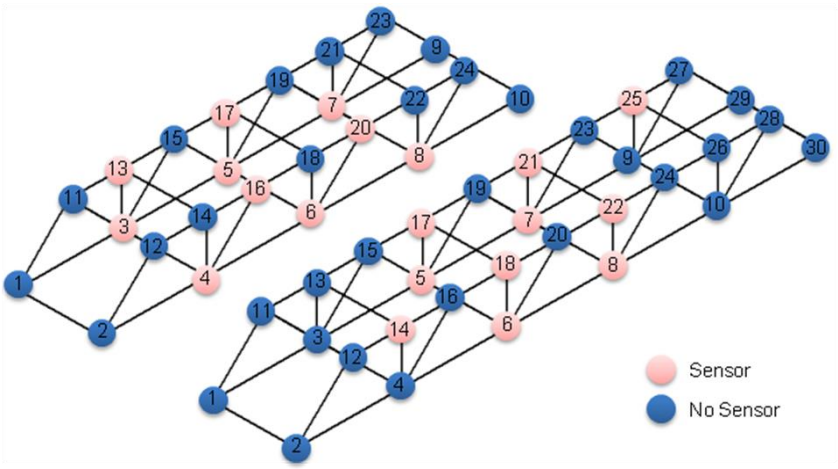


Figure 4-8 Sensor Layout of Short and Long Bridge Compositions

To enable vertical and lateral acceleration measurement, two sensors were placed at each identified node using the installed magnetic mounts. This instrumentation plan resulted in a total of ten different nodal locations to be monitored each collecting horizontal and vertical response data. The extended bridge model increased the number of nodal locations by from 24 to 30. To capture the general response of the extended structure, the accelerometers were relocated to alternate locations as shown in Figure 4-8. The installation of lateral cross bracing is not shown in Figure 4-8 as it did not influence sensor placement.

Impact test. There are several methods for conducting vibration tests and, as discussed in Chapter 2, the use of an impact hammer to impose vibrations to a test specimen is a quick and common approach. The purpose of the impact tests on the bridge was to obtain initial dynamic parameters of the system in a timely manner. For the most accurate and ideal system identification techniques, an impact hammer

equipped with a tip load cell tip may be used to document a known input energy level. For impact tests of the bridge models, a mallet hammer without load cell instrumentation was used to impart forces to excite the structures. Due to the multi-dimensional response of the model, modes in the horizontal, vertical, and torsional directions were anticipated. As a result, a series of impact tests were applied both horizontally and vertically, allowing for the dominating mode shapes to be present for the direction in which the structure was impacted. For example, in a horizontal impact test, the dominating response data will be derived from node accelerations in the horizontal direction; therefore, subsequent modal analysis should contain higher energy at these horizontal frequencies. Shown below in Figure 4-9 is an example of frequency domain response data after being subjected to a horizontal impact test.

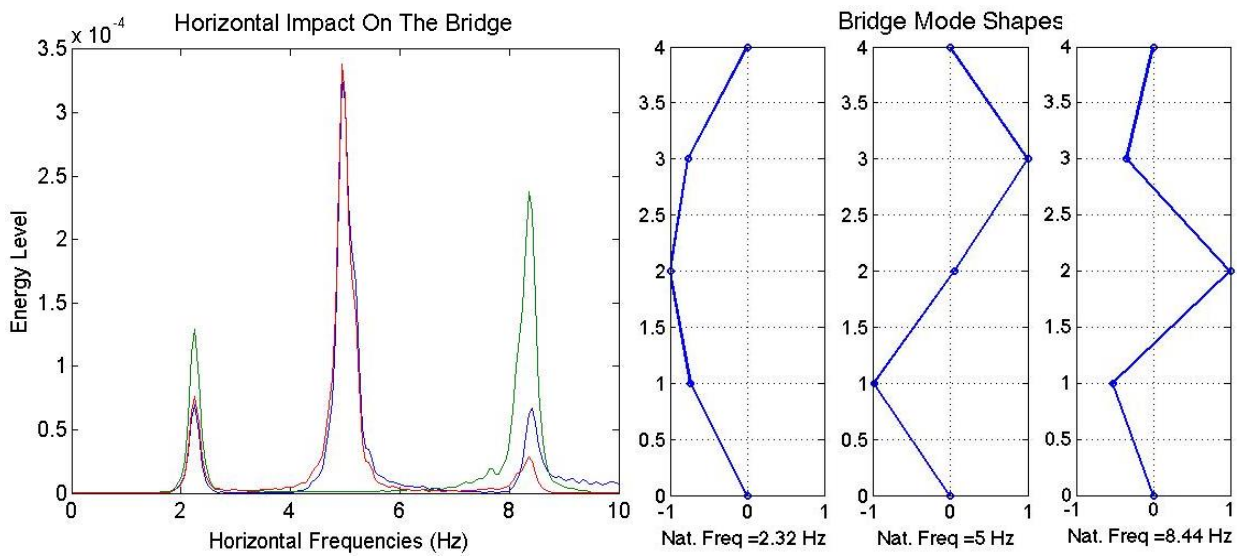


Figure 4-9 Horizontal Impact Frequency Response and Mode Shapes

Using the “peak picking” method to estimate the natural frequencies of a structure it is shown that the first three horizontal natural frequencies are 2.32Hz, 5.00Hz, and 8.44Hz. By following the steps below, the mode shapes illustrated in Figure 4-9 were able to be obtained:

1. Select a reference node. This is typically a node of impact.

2. Calculate the cross power spectral density (PSD) function (a frequency domain representation) between each measurement node and the reference node.

3. Identify peaks in the frequency response (PSD) that are believed to represent a natural frequency.

4. Determine the amplitude of the PSD at each natural frequency

5. Analyze the phase angle value of the cross PSD at the selected frequencies to determine whether it's in or out of phase with the reference node

6. Construct the mode shapes by plotting the value of the PSD amplitude measured at each nodal location for a particular identified natural frequency, normalized by the maximum amplitude. Values are positive if they are in phase with the reference and negative if they are out of phase.

The mode shapes in Figure 4-9 correspond to laterally displaced shapes and are best visualized from a plan view (i.e. looking down on the bridge). The selection of the impact node location is determined by the anticipated bridge mode shapes. As shown in the second mode shape plot (corresponding to a natural frequency of 5 Hz) in Figure 4-9, the mid span node remains stationary while the quarter span modes have equal and opposite magnitudes. It is important to note that if impact was applied at the mid span location, minimal energy would be contributed to the second modal frequency and result in negligible response for that mode. Impact nodes of interest changed between horizontal and vertical impact tests. Successive experiments were conducted on the different test specimens to study the dynamic effects of both an extended and laterally cross braced variations of the model with experimental results compared in a later section titled "Experimental Results".

Ambient vibration test. A secondary excitation approach was also applied as forced vibration method, with the use of the long stroke shaker. For these models, the purpose of forced vibration testing was to provide energy input in a specific frequency

range in the vertical direction where the impact test was unable to provide adequate excitation. These tests were conducted by fixing a long stroke shaker (APS 113 Electro-seis) to a bottom connection block, Node 8, to apply a vertical random excitation. A band limited white noise signal was created with the use of a Lab View interface enabling the alteration of various signal parameters such as white noise amplitude and bandwidth cutoff frequencies controlling the excitation's upper and lower frequency content. Figure 4-10 displays example frequency response data of the vertical motion of the bridge when exposed to a band limited white noise signal between 20 and 120 Hz.

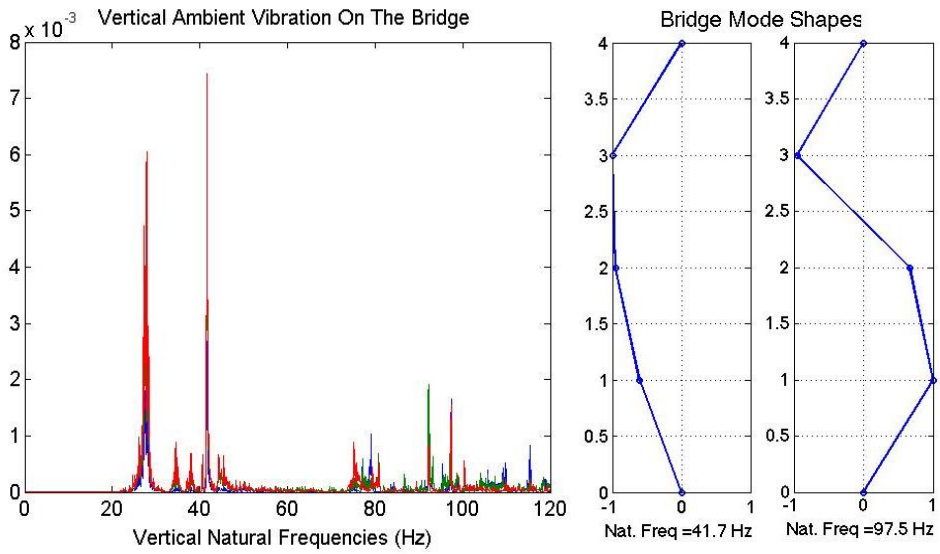


Figure 4-10 Band Limited White Noise Frequency Response and Mode Shapes

Due to the high levels of stiffness in the vertical direction it was difficult to impose adequate energy into the structure to excite higher energy modes and as a result limited the level of response vibration. As a result, the frequency response data shown in Figure 4-10 has a low signal to noise ratio in comparison to horizontal impact response data (Figure 4-9). Even when plotted in a logarithmic scale, where frequency peaks are more easily identified, differences in peaks are difficult to distinguish. The cause of the

challenges associated with distinguishing clear peaks in the response were investigated and further explained later in the chapter.

Due to configuration and space limitations, there was no practical method to utilize the long stroke shaker for horizontal excitation; the excitation in the horizontal direction was strictly limited to the use of an impact hammer. This constraint clearly illustrates how the experimental limitations of a dynamic research laboratory are derived from many factors, such as equipment capabilities and space constraints. .

In contrast to an impact test where only a hammer is used to generate excitation, a forced vibration test by use of shaker equipment contains many steps in the process of signal generation and application. Figure 4-11 breaks down the ambient vibration test set up into a simplified process as it pertains to this facility and identifies each component in the order in which it applies.

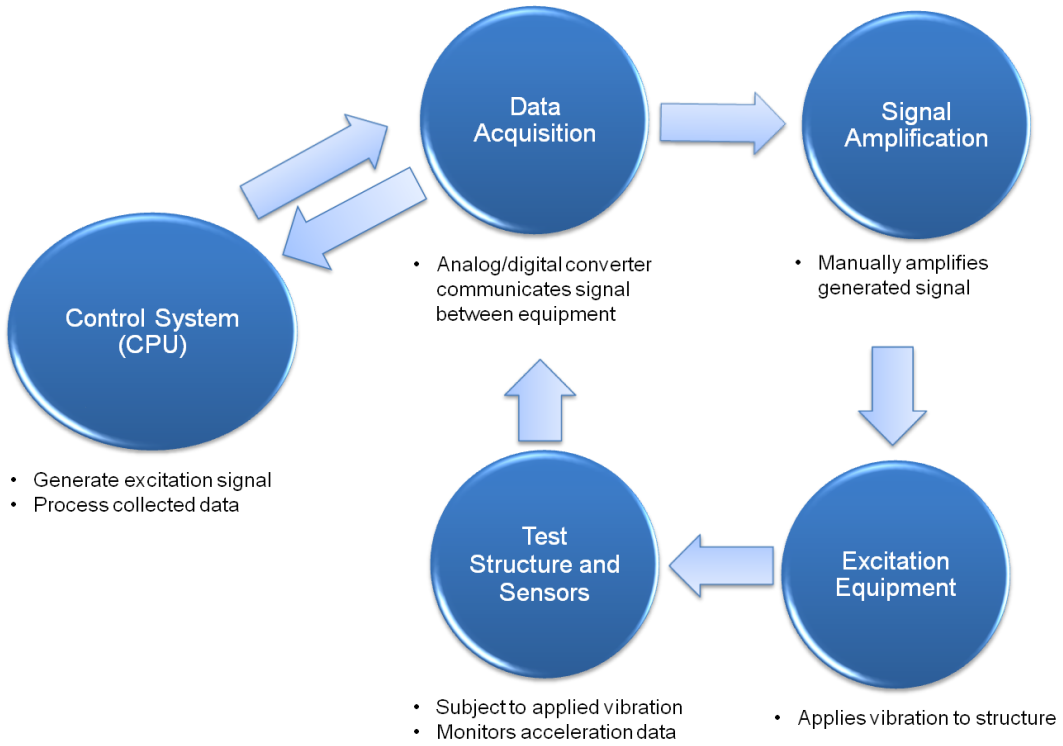


Figure 4-11 Shaker Test Process

In a forced vibration test, the signal is generated through the control system such as a LabView interface. Input parameters are manipulated according to the type of excitation the test specimen is to be subject to. The signal is then sent to the data acquisition system (DAQ) where it is converted from a digital to analog signal to be sent to the amplifier and shaker. The amplifier (APS Amplifier 125) is used to scale the signal to the desired magnitude. Manual adjustments are used to modify the proportional gain and voltage of the incoming signal which is then sent to the vertical shaker to impose vibration to the test structure. Magnetically mounted accelerometers located at discrete degrees of freedom collect vibration data that is then sent back to the DAQ for signal conversion back to the control system. Digital data is then stored to be used for data post processing and modal analysis. This basic dynamic testing process is similar in most structural dynamic research facilities, and although the stages have been shown at distinct phases and grouped as they relate to the facility in this work, it should be noted that these processes can be incorporated in as little a single device (Salawu and Williams 1993).

It must be noted that vertically applied motion to the structure resulted in frequency response peaks that were difficult to identify and associate with resonant frequencies. This is due to the overall high stiffness of the bridge models in the vertical and horizontally braced directions. In the case of forced vibration, the origin of this matter is a combination of the shakers capabilities and inherent bridge stiffness. In a band limited white noise signal, equal energy is provided for all frequencies in the specified bandwidth range therefore a very wide range of frequencies will result in lower energy input. The velocity, power, and stroke limits of a shaker pose a challenge to

provide sufficient energy to significantly excite a mode at a higher frequency value with a large enough force to capture vibration response. This is the case in the vertical direction of the bridge. Nodal amplitudes of vibration are limited due the inability for the shaker to apply enough force at a high frequency range in which the vertical frequencies of the bridge reside. As a result, a low signal to noise ratio makes it difficult for clear peaks to be detected in the frequency domain.

Figure 4-12 is a graphical representation of the shaker capabilities. This illustrates the maximum force envelope for a fixed body of the APS 113 shaker is 133N (30lbf) which is only attainable from 0 ~ 25 Hz, after which an apparent decrease in force potential develops.

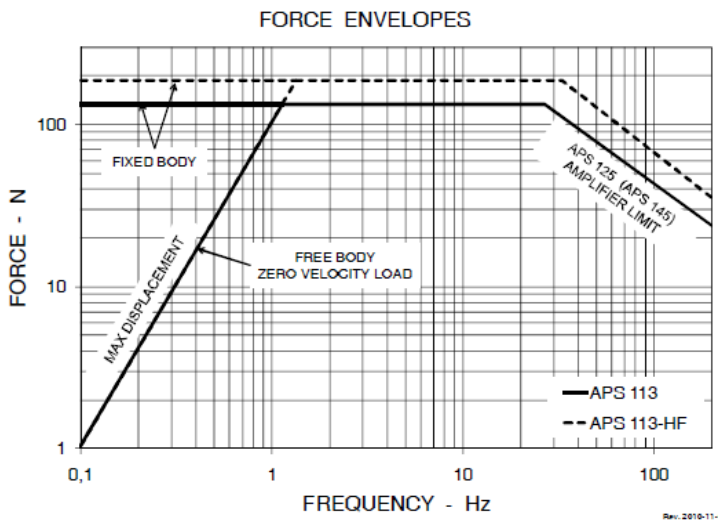


Figure 4-12 APS 113 Performance Curve (APS Dynamics)

A static analysis revealed a maximum displacement of 0.004 inches when a 30lb force was applied to the structure, a displacement too small to obtain the clear dynamic characteristics desired from an experiment. The intended functionality of the APS 113 Shaker is for dynamic experiments but at a relatively low frequency range. For the

bridge models, it can be concluded that is not a suitable excitation technique for vertical vibration applications.

Test Results

This section will present response data from the dynamic experiments conducted with both hammer impact and ambient vibration applied by a vertical shaker, followed by a comprehensive summary of experimental results.

In instances where the bridge stiffness levels were of high magnitude, it was difficult to identify clear peaks to be associated with modal frequencies in the frequency response data. In these cases, the predicted natural frequencies from the finite element model were used as a point of reference to identify candidate peaks allowing for a guided estimate of dynamic response.

Model (1) Horizontal. The original bridge orientation had a span of ten feet without lateral bracing. As a baseline measure of the unmodified model, horizontal impact tests were conducted and response vibration was monitored. Horizontal natural frequencies of the model (1) were determined using a PSD function and plotted in the frequency domain (log scale) shown in Figure 4-13.

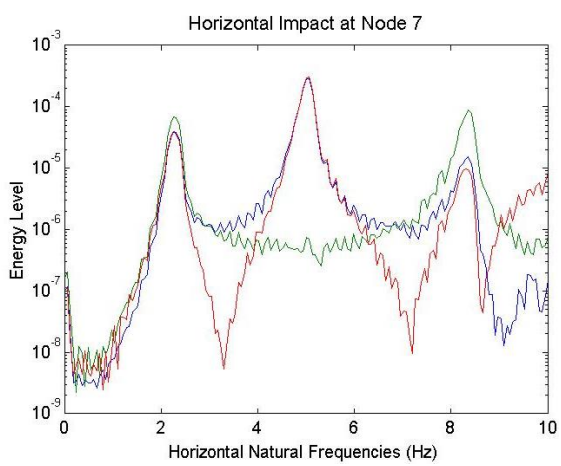


Figure 4-13 Horizontal Impact Response of Model (1)

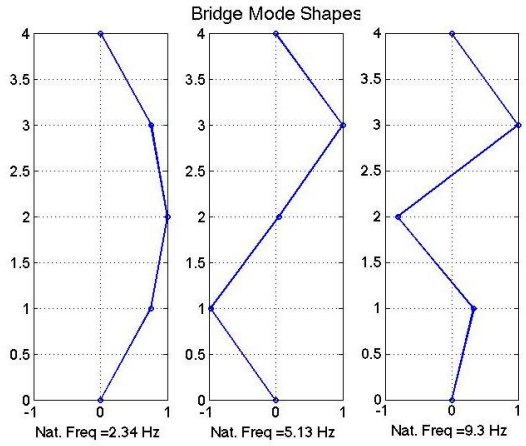


Figure 4-14 Horizontal Mode Shape of Model (1)

The dominant peaks at 2.34 Hz, 5.13 Hz and 9.3 Hz are linked to the first, second and third horizontal resonant frequencies of the bridge specimen, respectively. This unmodified version of the bridge model provided a responsive structure for which to conduct structural dynamic experiments and straight forwardly apply the peak picking technique to obtain modal identities. The finite element model predicted the first three horizontal frequencies to occur at 5.9, 13.0, 20.7 Hz. It is observed that the FEM values are an average of 2.5 times greater than those presented in the data in Figure 4-13, indicating either a lack of mass representation or excessive stiffness representation in the finite element model. With the high confidence level in the experimental horizontal response, this FE to experimental model relationship can be recorded and potentially applied to help characterize the bridge response in other modifications.

Model (1) Vertical. To isolate response in the vertical direction, random vibration was applied using the electromagnetic shaker. The projected frequencies from the finite element model resulted in a first vertical mode at 60.1 Hz, so a white noise signal with bandwidth ranging from 20-100Hz was used to target the first vertical natural frequency

of the bridge. Shown in Figure 4-15 is the vertical frequency response data from a vertically applied ambient vibration from the long stroke vertical shaker.

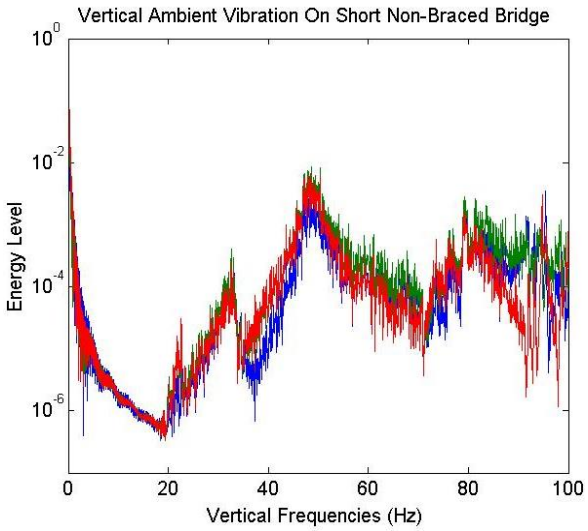


Figure 4-15 Vertical Response from Shaker of Model (1)

In an attempt to improve the low signal to noise ratio of bridge response shown in Figure 4-15, hammer impact was applied downward on the model (1) with the intention of providing sufficient energy to excite a vertical mode. Figure 4-16 shows the bridge response plotted in the frequency domain as a result of vertical impact on the bridge.

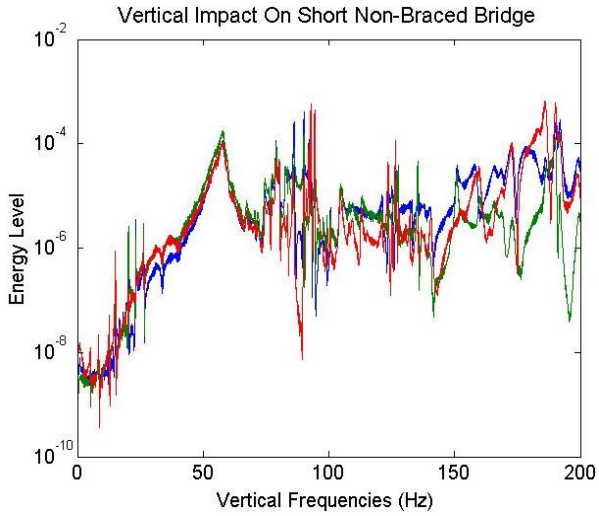


Figure 4-16 Vertical Response from Impact of Model (1)

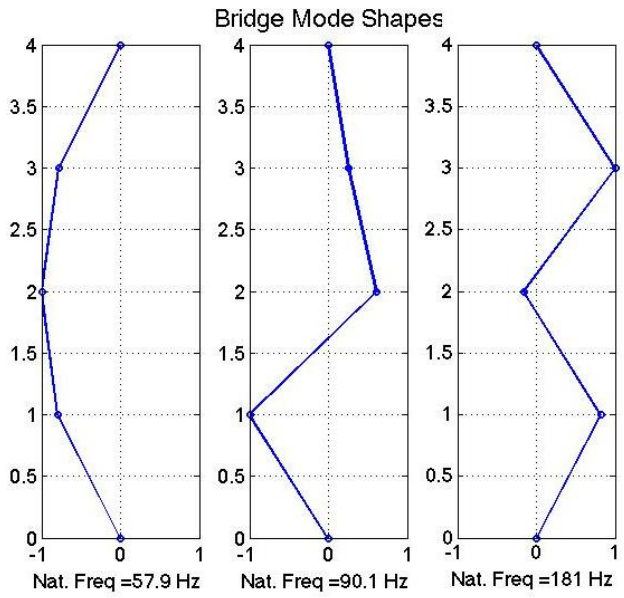


Figure 4-17 Vertical Mode Shape of Model (1)

A persistent low signal to noise ratio due to the lack of energy applied to the structure makes it difficult to confidently identify the frequency peak associated with a particular mode. The most evident peak appears at 57.9 Hz and can be concluded to be the first bending modal frequency in the vertical direction.

As previously identified, FEM predictions indicated the first vertical frequency to occur at 60.1Hz which is relatively close to first natural frequency of 57.9Hz. A new FE to experimental model relationship was then developed to assist in the interpretation of noisy vertical response data. Subsequently, a 2nd and 3rd vertical mode could be chosen, however the irregular shape of mode 2 illustrated in Figure 4-16, is a reminder that the 2nd and 3rd natural frequencies are strictly estimated values.

The experimental results from model (1) in both the horizontal and vertical directions, proved to be critical information to compare with the finite element model predictions. Using the FE predictions as a reference and understanding how those

predictions relate to the experimental model, reasonable estimates can be applied to future response data sets to identify modal parameters of additional bridge models.

Model (2) Horizontal. The inherited configuration of the bridge was experimentally proven to be overly flexible in the horizontal direction, as compared to a typical full-scale bridge. As an initial modification, lateral bracing was added to the bottom bridge deck of the structure preserving the ten foot span but adding lateral force resistance to the system. Figure 4-18 displays response from horizontal impact tests.

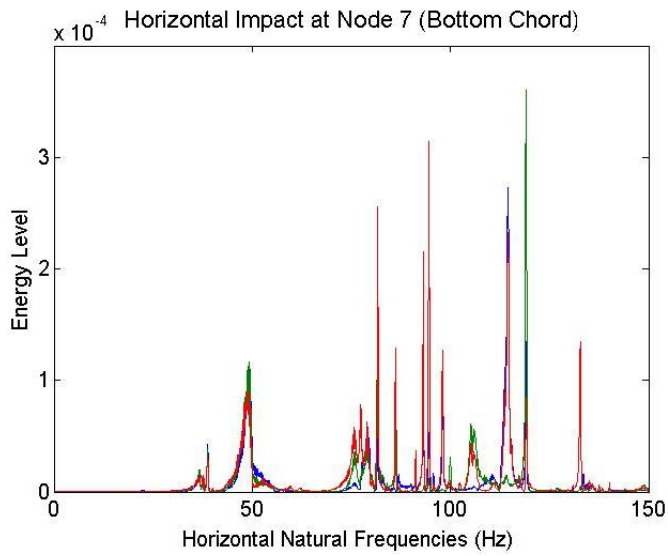


Figure 4-18 Horizontal Response from Impact of Model (2)

Various peaks occur throughout the frequency response plot and it is difficult to attribute one specific peak to a horizontal mode. FEM predictions of the first three horizontal modes for model (2) are 93.8, 197.9, and 204.8Hz. Using the previously established horizontal relationship between the experimental and FE model frequency values, an estimated approach to determining the horizontal frequencies of model (2) can be applied. Figure 4-19 is the mode shape plot from the frequency response data found in Figure 4-18.

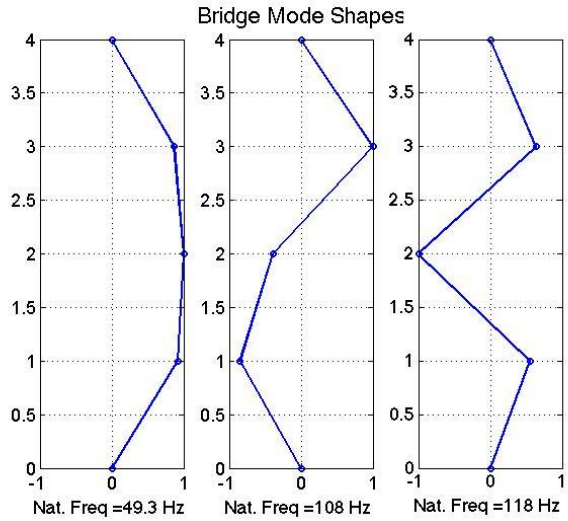


Figure 4-19 Estimated Horizontal Modes of Model (2)

As a result, the first three estimated horizontal frequencies of model (2) are 49.3, 108, and 118Hz, respectively. In instances where numerous peaks occur in a frequency response such as the data shown for model (2), the use of a FE model can be a useful tool in estimating dynamic response.

Model (2) Vertical. Vertical response is plotted in Figure 4-20 and Figure 4-21 as a result of vertical shaker and hammer impact tests, respectively.

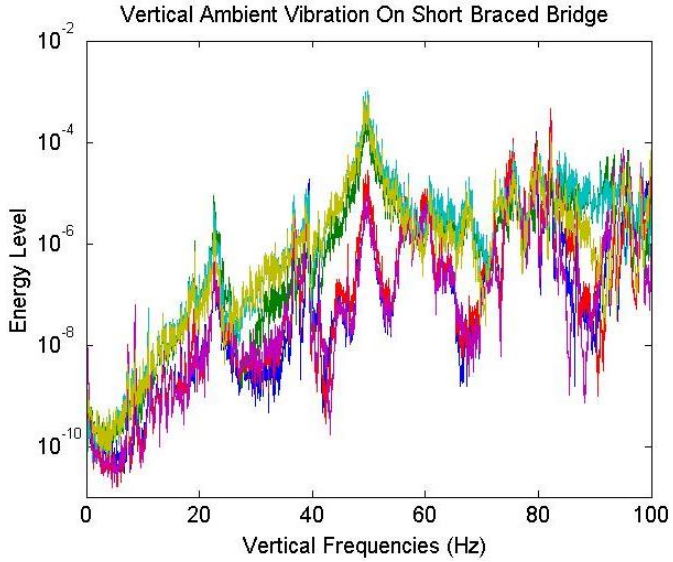


Figure 4-20 Vertical Response from Shaker of Model (2)

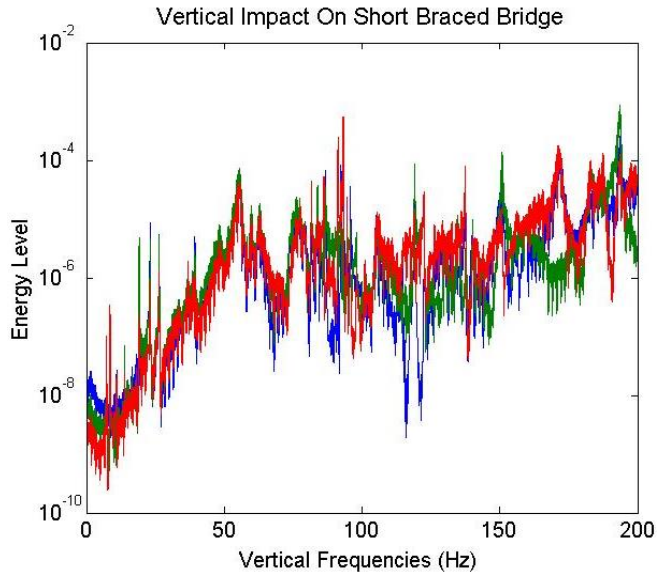


Figure 4-21 Vertical Response from Impact of Model (2)

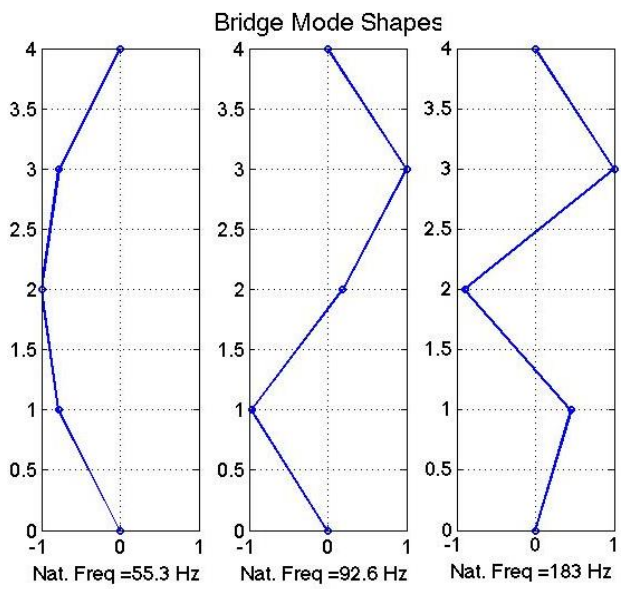


Figure 4-22 Estimated Vertical Modes of Model (2)

As in model (1) vertical data, model (2) response data is filled with peaks at many frequency ranges at similar amplitudes. In model (1) peaks were difficult to identify and now with the addition of lateral cross bracing, more members are included in the system and present an even noisier data set. Selection of peak values for this test set was based on previous vertical test results since the addition of horizontal lateral bracing

should have minimal effect on the vertical response of the system. As a result, mode shape plots in Figure 4-21 mimic typical shapes for mode 1, 2, and 3.

Model (3) Horizontal. A final modification was the extension of the braced bridge model (2). This extension added an extra 2.5 foot bay extending the total span to 12.5 feet and preserved the lateral bracing components. Figure 4-23 shows model (3), the current composition of the bridge with vertical shaker attached.

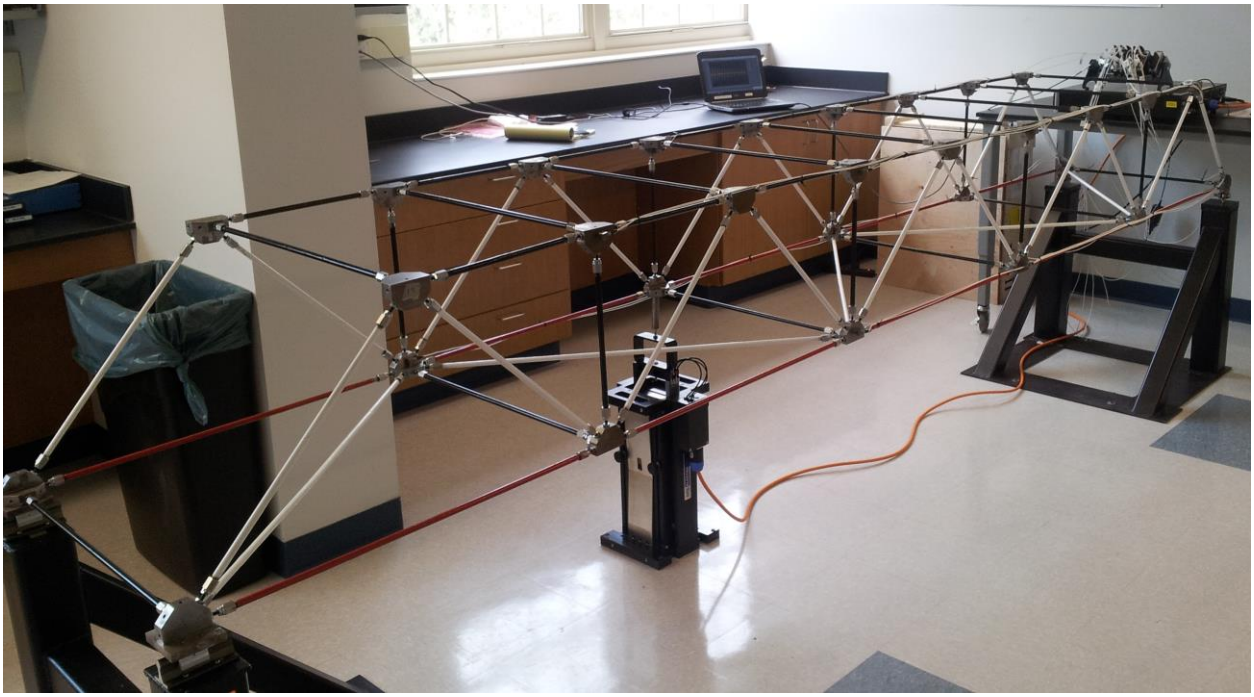


Figure 4-23 Current Bridge Configuration

After evaluation the FE model predictions (Table 4-1), it was determined that lengthening the bridge model would not affect the horizontal flexibility of the bridge as much as it would the vertical direction. The first vertical mode is expected to drop by roughly 20 Hz from model (2) to model (3). The dynamic response from the braced extension is illustrated in the figures below.

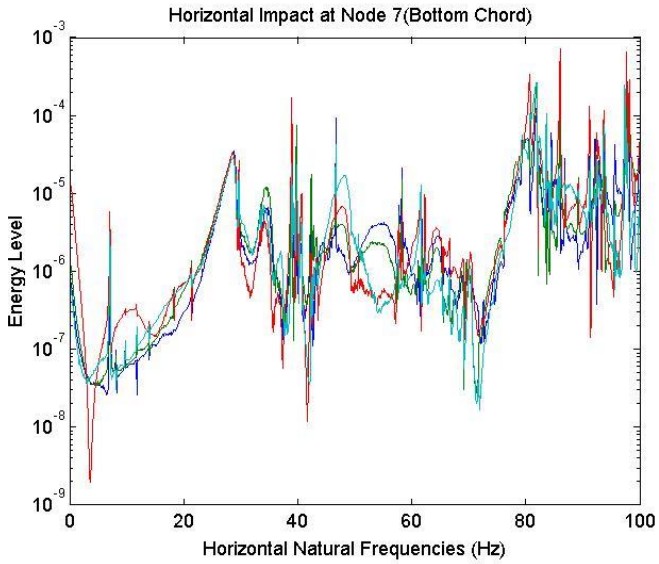


Figure 4-24 Horizontal Response from Impact of Model (3)

FEM predictions of the first three horizontal modes for model (3) are 90.7, 194.0, and 197.1Hz. Preserving the established horizontal relationship originally developed with model (1) an estimated approach can be applied once more to estimate the horizontal frequencies of model (2). Horizontal mode shapes and their associated frequency values are shown in Figure 4-25.

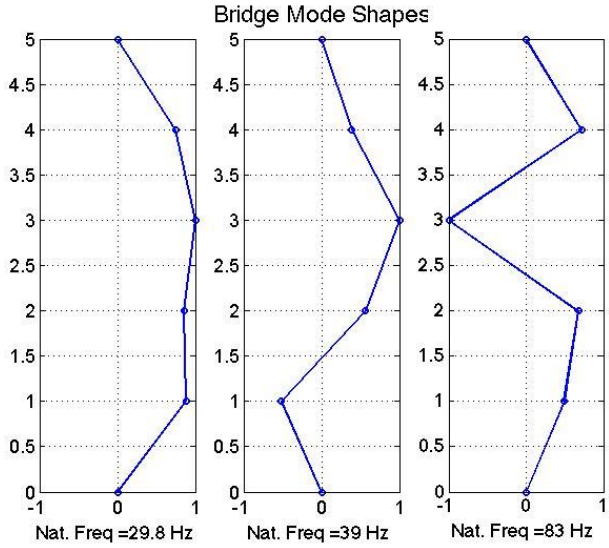


Figure 4-25 Estimated Horizontal Modes of Model (3)

The first three estimated horizontal frequencies of model (3) are 29.8, 39.0, and 83.0Hz, respectively. Although the addition of new horizontal elements creates a more complex structure, the use of a FE model to estimate experimental response can be adopted as a reasonable approach to estimate dynamic characteristics.

Model (3) Vertical. The vertical frequency response is plotted in Figure 4-20 and Figure 4-21 as a result of vertical shaker and hammer impact tests, respectively.

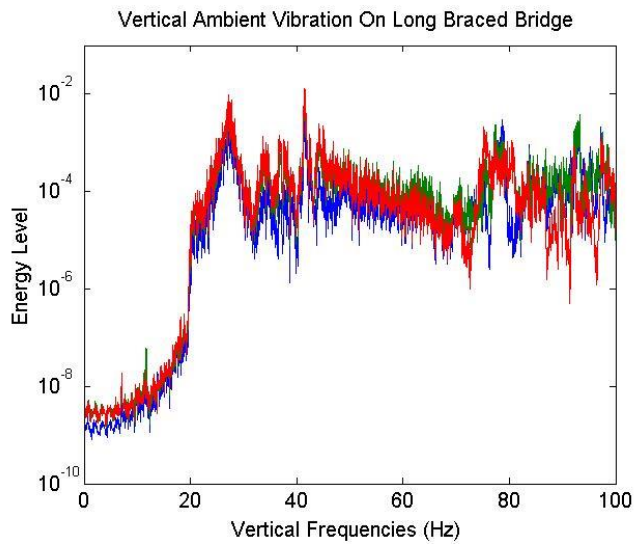


Figure 4-26 Vertical Response from Shaker of Model (3)

Frequency response data of the extended and horizontally braced bridge model is illustrated in Figure 4-26. This band limited white noise signal was generated with frequency content ranging from 20-100Hz and explains the “ramp” up of energy at the 20Hz mark. Similar to past shaker applications on the bridge, there is very little energy differential and separation between peaks. A relatively noticeable peak is apparent in the interval of 20-40Hz however it is impossible by visual inspection to determine what this peak represents. Vertical impact applied at Node 21 is shown below in Figure 4-27 but has similar signal to noise ratio issues.

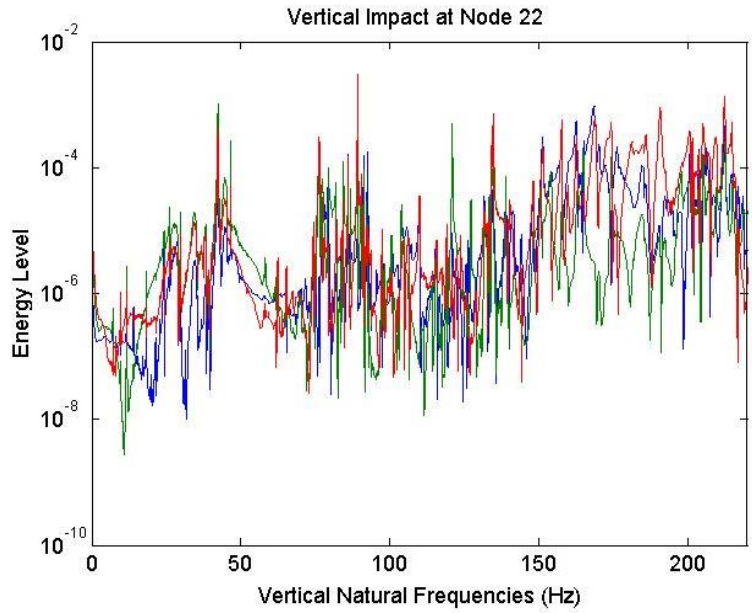


Figure 4-27 Vertical Impact Response of Model (3)

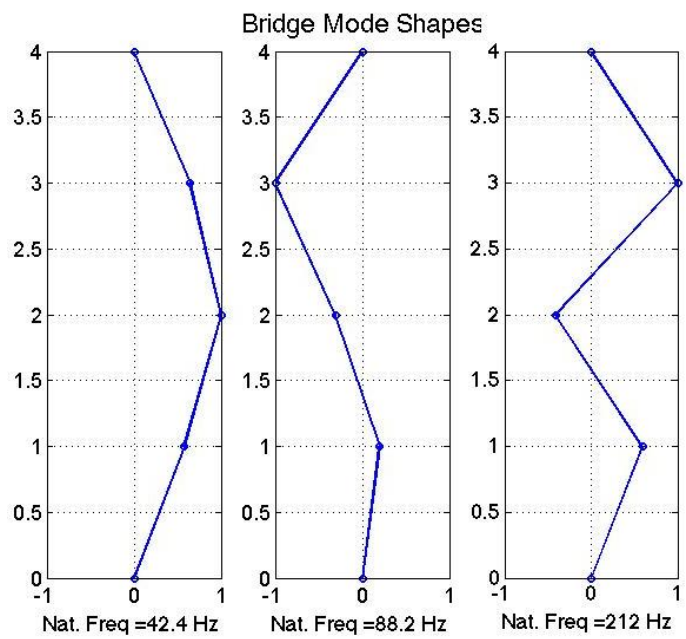


Figure 4-28 Estimated Vertical Modes of Model (3)

By applying a vertical impact to the test structure, only a percentage of the frequency range for which this structure responds is excited. Both horizontal and vertical

directions are braced in their respective planes and cause an overall increase in system stiffness and subsequent natural frequencies.

Predicted vs. Experimental. Table 4-2 presents estimated experimental natural frequency values of all models along with their predicted FE values for a global comparison.

Table 4-2 FEM vs. Experimental Natural Frequencies

Mode Shape	Frequency (Hz)					
	Original		Short Braced		Long Braced	
	FEM	Experiment	FEM	Experiment	FEM	Experiment
Horizontal						
1st	5.9	2.34	93.76	49.3	90.70	29.8
2nd	13.0	5.13	197.9	108.0	194.3	39.0
3rd	20.7	9.3	204.8	118.0	200.1	83.0
Vertical						
1st	60.1	57.9	60.9	55.3	41.1	42.4
2nd	107.0	90.1	93.5	92.6	92.3	88.2
3rd	204.0	181.0	202.4	183.0	196.7	212

Although the FE model predictions were used as a guideline to select candidate peaks in the frequency domain, the discrepancy of the FEM vs. experimental results needs to be addressed. As mentioned, the horizontal predictions for the FE model were 2.5 times higher than the resulting experimental horizontal response. In the vertical direction the relationship established showed a very close comparison between the FE model and experimental structure. In any creation of a computer based model there are a number of assumptions that are made on member connectivity and boundary conditions. This difference in the FE model can be attributed to an overestimation of rotational stiffness at supports or at nodal locations. As a future recommendation, model updating is an encouraged path for future work with the FE model to create a

model that can accurately predict the response of the structure in both the horizontal and vertical directions.

The data analysis challenges presented in this section corresponding to bridge stiffness and excitation equipment limitations are important factors that must be considered when selecting both a new structural model and excitation equipment for a dynamic research lab. The combination of high bridge stiffness in the vertical direction and the inability for the vertical shaker to impose appropriate force at the high frequency range of the structure made it extremely difficult to confidently characterize the dynamic response of all bridge models. It is recommended that if future work is to be conducted with this model where natural frequencies in all directions need to be accurately quantified, then alternate excitation methods must be explored. Table 4-3 provides a list of shakers that can be implemented in future testing applications.

Table 4-3 Alternate Shaker Systems

Shaker Type	Freq. Range (Hz)	Max Force (kN)	Max Force (lbf)	Max Disp. (in)	Bridge Stress (ksi)
APS 113	0-200	0.133	30	0.004	0.76
Modal Exciter	2-5000	1	225	0.028	5.73
Low force	0-4000	5.12	1150	0.15	29.28
Medium force	0-3500	9.8	2200	0.28	56.02
Medium force	0-3000	22.2	5000	0.63	127.32

The current shaker model is listed above as APS 113 and has a capacity of producing a maximum of displacement of 0.004in on the current bridge model (3). With the selection of an alternate shaking device, larger displacements can be generated and subsequent modal analysis performed. The applied bridge stress column to the far right of the table is also included to the resulting stress applied to the bridge at this forces is known and a yielding failure mode is not encountered.

Although natural frequencies ranges are relatively high compared to practical applications, this bridge specimen can still be used for damage detection applications by replacing truss members with other members possessing a decreased cross sectional area as described by Hernandez 2011. If a newly adopted shaker cannot be purchased due to cost limitations, this structure can be modified to induce a decrease in overall system stiffness. This goal could be accomplished by applying a permanently distributed mass to the structure or additional modification to the connection blocks to influence the vertically oriented diagonal truss members. Manipulating the angles of these members in a manner to reduce the total number of members resisting vertical forces will lead to a decrease in vertical stiffness. After these modifications, this bridge model will be a multi-purpose test tool and will add to the inventory of structures presented further in this work.

2–D Dynamic Building Model

Structural models integrated in a dynamic research facility are done so to contribute to a variety of testing applications. The structural bridge model presented in the previous section is a useful tool but for very specific testing situations. To extend the capabilities of the laboratory identified in this work, a seven-story, two-dimensional structural dynamic building model was designed and fabricated. This section provides a detailed explanation on the design, construction, and testing of the building model. The purpose of this structure was to introduce a building test model that is responsive to low energy excitation that possesses clearly identifiable, well-separated natural frequencies. Such a model can be used as a bench mark structure and apply to a number of current and future research projects.

Design

This section will explain the rationale behind the final building design and review the dynamic analysis. With no particular parent structure on which to base the design, physical facility constraints were used to provide initial design bounds for overall model dimensions, applied mass, and boundary conditions. Figure 4-29 identifies width (W), height (H), mass (M), fixity (k) and material/cross section (EI) as critical variables included in the building design process that influence the models dynamic response.

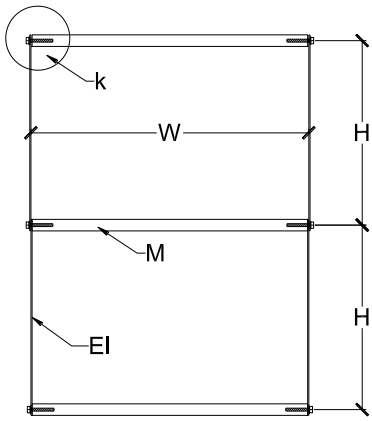


Figure 4-29 Building Variables

Model variables (H) and (W) were influenced by physical dimensions of both the lab facility and of the excitation equipment planned to be used for dynamic experiments. The platform dimension of the uniaxial shake table was used as a starting point for the width (W) of each floor of the building model and limited values to be no more than 18 inches. The shake table was mounted on strong table support allowing for 5 feet of clearance between the shake table elevation and an accessible height for sensor installation. Using the dimensional constraints provided by the laboratory and shake table, a width of 12 inches was selected and after concluding on a total of seven story levels, an inter story height of 8 inches was chosen. These parameters resulted in a

width two thirds the maximum limit. With evenly distributed dimensions of 8 inches per story, a total height resulted in 56 inches, half a foot under the maximum allowable height.

Material/cross section geometry (EI) and fixity (k) selection were based on the possible materials and connection configurations. The purpose of this structure is to provide a responsive model where dynamic characteristics are intended to be easily obtained. After investigating past experimental building models applied for structural dynamic research, a common column material used was spring steel allowing for a more flexible structural model. PVC was chosen as the floor material where mass would be affixed. With PVC material used at each floor level, spring steel was fastened by a tight bolted connection and similar to practical connection types, rotational restraint would fall between a structural 'fixed-fixed' and 'pin-pin' connection.

It was determined that the desired range of frequencies for the building would be representative of a commonly constructed multi story light frame structures, 0-20Hz. In this design, the bulk of the mass (M) would be concentrated at each floor, or each degree of freedom (DOF). With a total of seven stories, this system was modeled as a 7th order multi-DOF flexible "shear" building frame and a preliminary modal analysis (Appendix A) based on MDOF equation of motion principles (Eq. 2-4) was implemented for design. Listed below are the concluded design parameters corresponding to the variables identified in Figure 4-29:

- E = 30,000ksi
- I = 0.000031250 in^4
- H = 8 in.
- W = 12 in.
- k factor = 12 (assumed rigid)
- M = 4.2 lbf

By adjusting the applied mass, holding dimensions, material, and boundary conditions constant and solving the equation of motion, frequencies could be predicted and targeted to the desired range prior to model construction/testing. The preliminary predictions of the system natural frequencies are shown on Table 4-4.

Table 4-4 Predicted Building Natural Frequencies

Mode	Frequency (Hz)
1	2.1
2	6.3
3	10.1
4	13.5
5	16.4
6	18.5
7	19.8

The calculated frequencies obtained from the dynamic analysis explained above are compared to experimental frequencies later in this section. Figure 4-30 identifies the design parameters of the final building design to be used for the fabrication phase.

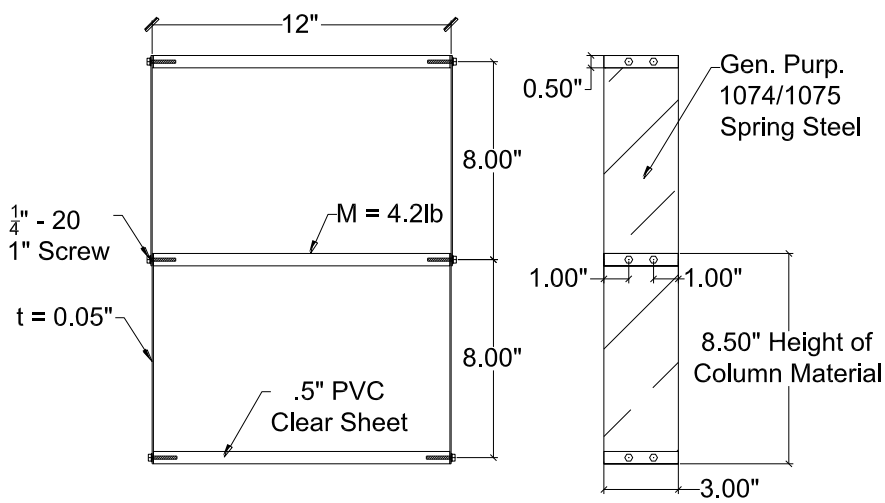


Figure 4-30 Final 2-D Building Design

Two of the seven floors of the building system are illustrated above. The small size of the building model members and components made in-house fabrication a cost effective

and manageable option. The following section will outline both the fabrication and assembly process to construct a dependable model reflective of design parameters.

Fabrication

In this section, the fabrication methods of the two-dimensional building model are identified in detail. Maintaining accurate fabrication and assembly technique is critical during the construction of any structural model and allows for more confidence in experimental findings. Similar to the bridge assembly, individual tools used as well as precision methods will be presented as a reference for any future researcher wishing to construct a similar model.

Both column and floor plate material was ordered in bulk and trimmed to the appropriate dimensions noted in the design drawings (Appendix A). Columns and floor girders were cut from a 0.05 inch thick sheet of multi-purpose spring steel 1095 and a 0.5 inch thick clear PVC sheet, respectively. This was performed with a “vertical band saw machine” for the PVC floor plates and “horizontal foot shear” for the metal columns. Both machines were provided by the University of Florida Structures and Coastal laboratories and when combined with preliminary dimension marks resulted in a quick, economic, and accurate method to mass producing the 14 columns and eight floor plates illustrated in Figure 4-30. Steel plates were used to provide the isolated masses located at each story level. Using a horizontal band saw and noting a steel density of 0.282 lb/in^3, a 0.5” x 3” sheet of steel was ordered and cut into seven 8 inch, 4.2 lb. segments to meet the design guidelines.

The fastener layout called for ¼ inch bolt holes located at the top and bottom of all metal columns to be centered and separated by 1 inch (Figure 4-30). Additional ¾” through holes were needed at the center of each PVC floor beam and steel plate to fix

each mass at the floor level. All bolt holes were performed using a drill press machine with appropriate drill bit coupled with center punch device to assure proper hole placement. Once all components were cut and drilled to correct dimensions the erection phase required minimal forecasting. ¼”-20 fasteners tightened into predrilled holes connected PVC beams to the top and bottom of each steel column. This member orientation was preserved for seven stories; after which mass was applied to each level and fastened with a ½ bolt. Figure 4-31 shows the erected structural model after the final stages of assembly.

Figure 4-31 Erected 2-D Building Model

With the proper fabrication and assembly methods, this two-dimensional building model will prove to be a valuable structural dynamic research tool to be used in numerous experiments and academic applications.

Dynamic Testing and Results

Although the model's construction and dimensional layout closely matched design requirements, physical fabrication of structural members and various assumptions in analysis can introduce discrepancies between the actual and predicted response. As a result, with any new structural model, dynamic experiments must be conducted in order to characterize inherent dynamic properties and quantify its response to dynamic loading. Similar to the bridge model discussed earlier in this work, a series of hammer impact and forced vibration experiments were used to input energy to the system and collect structural response data to obtain natural frequencies and mode shapes of the building model. This section identifies the sensor instrumentation and dynamic testing details of both impact and random ground excitation methods.

Instrumentation. To capture building response to dynamic excitation, the uniaxial accelerometers (PCB Piezotronic ICP) joined with the 20-channel data acquisition system (National Instruments CompactDAQ) previously implemented on bridge tests were applied. Seven degrees of freedom combined with a two-dimensional building orientation resulted in a total of seven accelerometers installed; one at each floor level of the building. The magnetic feature of the sensors allowed for easy attachment to the exterior of each column. Figure 4-32 is a schematic of the sensor layout for the building model.

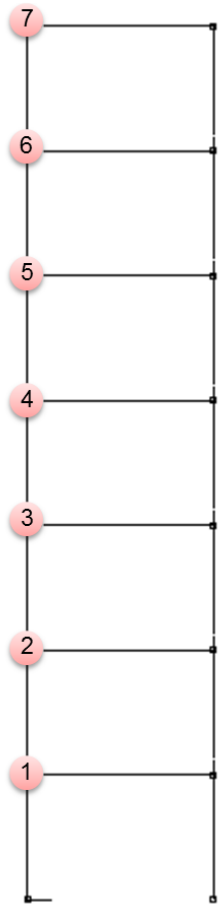


Figure 4-32 2-D Building Sensor Layout

In contrast to the bridge model, this building model only had one orientation with no additional modifications; therefore the manipulation in sensor layout was not needed and could be preserved for both impact and random vibration tests. Because the stiffness differential between rigid floor beams and slender columns is significantly high, only one chord of the system is needed to me monitored to accurately capture system response. As shown above, the string of nodes included on the left chord was chosen. During both impact and random vibration testing, the bottom PVC floor plate was fixed to the base floor to establish a rigid connection. It is common practice during a dynamic test to capture input energy being applied to the system so an eighth sensor was fixed

to the shake table platform during the shake table ground excitation tests. By capturing input vibration, future structural identification methods can be completed to obtain full dynamic characteristics of the structure.

Impact Test. Applying energy by use of an impact hammer was the initial excitation method used on the model and was a quick and effective testing method to verify model design goals. To perform this pretesting phase, a mallet hammer was used to provide impact to the test specimen. In-plane impact was the only applicable force direction due to the two-dimensional configuration of the model. Building frequency response data, as a result of hammer impact at the top floor, is displayed in Figure 4-33.

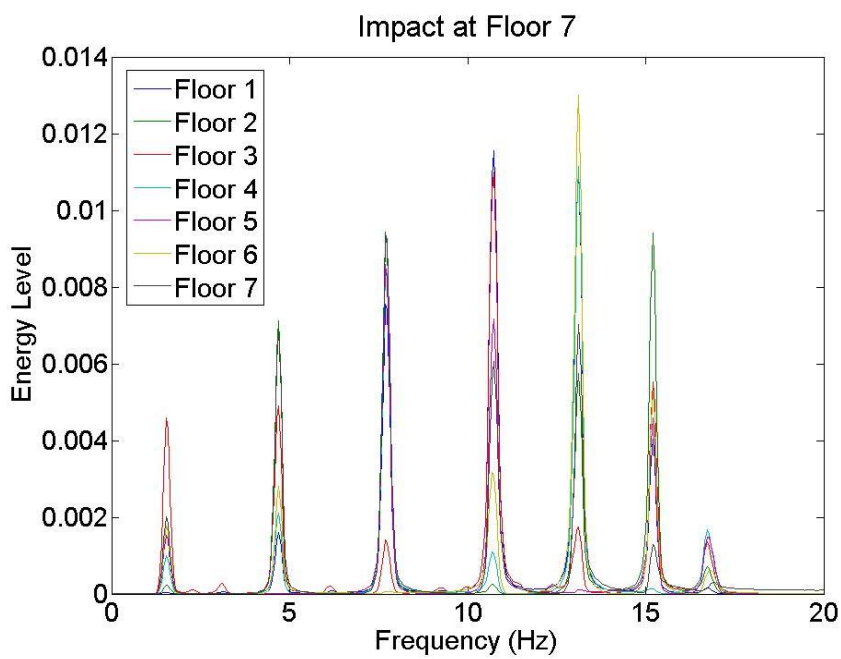


Figure 4-33 Frequency Response of 2-D Building from Impact

Plotted in the frequency domain, the seven fundamental frequencies can be clearly identified. Selection of nodal impact location for this structure was conducted in a similar fashion to previous bridge impact tests and relied on the expected mode shapes

of the structure. For example, maximum displacement in a Mode 1 dominating response occurs at the top floor level. When deciding on a floor of impact, if Mode 1 is the modal response of interest then impact should be applied to the top floor. If Mode 2 is the modal response of interest then impact should be applied the floor with the largest anticipated displacement for Mode 2 which is at the quarter points. It is important to note that if Mode 2 was the response mode of interest and impact was applied to the middle span of the structure, then minimal energy would be applied to Mode 2 because the node of impact has no theoretical vibration response.

Ambient vibration test. After exposing the building model to impact dynamic testing and obtaining resonant frequencies, the next step in the dynamic testing program was to apply forced ground excitation by means of the uniaxial shake table. This test method is used as a secondary measure to confirm structural dynamic findings previously obtained from the less complex impact tests detailed earlier. Subjected to a band limited white noise vibration, this structure was monitored in a similar fashion as impact tests conserving the same sensor type and orientation with the addition of a sensor at the base of the shake table platform. A large benefit to the impact pretesting previously described, is not only the identification of fundamental modes, but the confidence in the selection of frequency range for a random vibration test. As discussed earlier, a band limited white noise vibration signal requires a range of frequency content for which the random signal is to be generated. If dynamic characteristics of this test structure were unknown, an estimated frequency range would need to be chosen and then refined for future shake table tests. After obtaining the seven modal frequencies from the pretesting procedure, a frequency range of 0-20Hz

was selected and applied to generate a random input signal. Shown in Figure 4-34 is frequency response data collected as a result of ambient vibration.

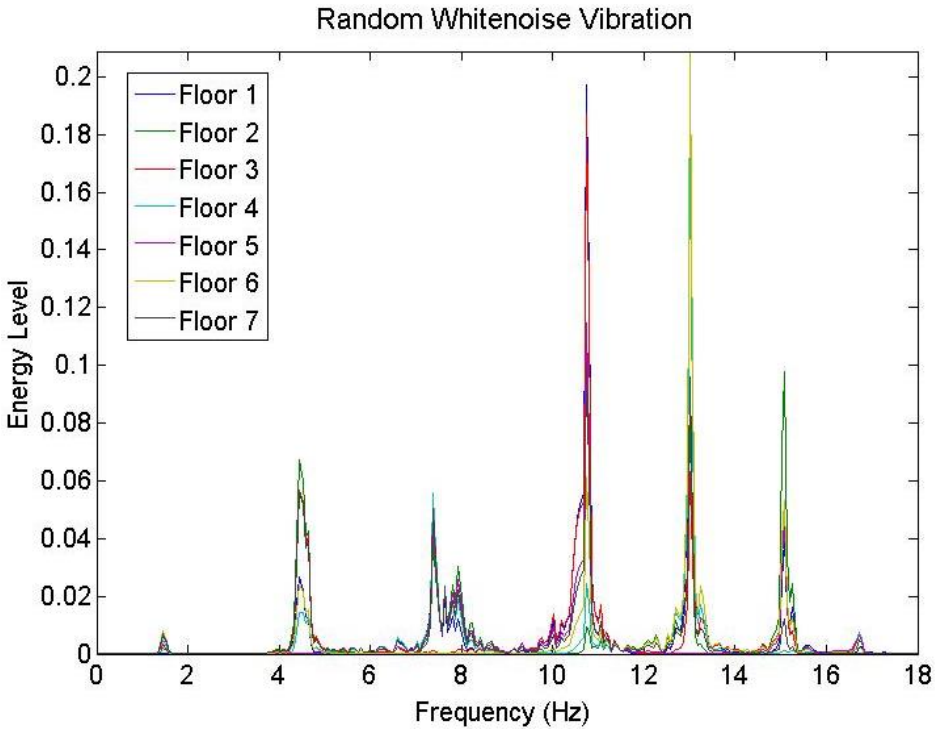


Figure 4-34 Frequency Response of 2-D Building

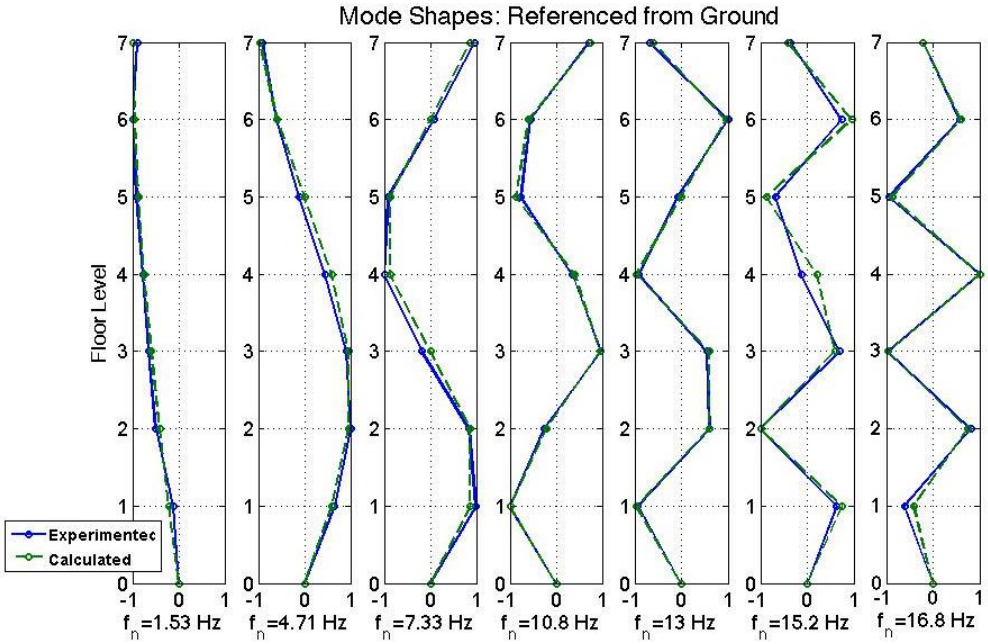


Figure 4-35 Theoretical vs. Experimental Mode Shapes

Confirming the dynamic characterization found from impulse tests, the shake table vibration tests proved to be another dynamic testing method to obtain critical information on the structural system. Illustrated on each mode shape plot are the corresponding calculated theoretical mode shapes based on the design parameters. Using calculated eigenvectors with a reference node located at the shake table platform, these shapes are able to be compared to experimental mode shapes obtained from frequency response data. It is clear the theoretical behavior anticipated by applying the equation of motion is an accurate analysis method to predict system response for this test structure. It is also found that the peak picking technique is a dependable method for determining modal frequencies due to the well-separated, high-amplitude peaks in the frequency response of the model. In some instances, the frequency response peaks can be associated with bending modes of different directions or even torsional modes as was the case with the bridge. By designing one layer of structural members intending to only resist load in one direction, the presence of other bending or torsional components are eliminated, providing a dependable structural model with clear data that can be confidently quantified.

Predicted vs. Experimental. Although mode shapes illustrated in Figure 4-35 share similar deflection orientations, numerical magnitudes of natural frequencies differ slightly between design and experimental natural frequencies. Table 4-5 compares the calculated natural frequencies from Table 4-4 to the experimental frequencies obtained from both testing methods.

Table 4-5 Predicted vs. Experimental Natural Frequencies

Mode	Frequency (Hz)	
	Predicted	Experimental
1	2.1	1.5
2	6.3	4.7
3	10.1	7.3
4	13.5	10.8
5	16.4	13.0
6	18.5	15.2
7	19.8	16.8

Input variables applied to the dynamic analysis method include mass (m), material properties (EI), building dimensions (H) and fixity (k). While the transition from design to fabrication can alter some of these parameters, the only variable not strictly regulated is the fixity between beam and column interface. An assumption in the preliminary analysis was a rigid connection with column stiffness calculated as

$$k_{rigid} = \frac{12EI}{H^3} \tag{4-1}$$

where (H), (E) and (I) correspond to column height, modulus and inertia respectively. A coefficient of 12 was the support coefficient used to calculate the stiffness of a column with fixed ends which prevents rotation. In a case where top and bottom column supports possessed pinned ends, a coefficient of 3 would be applied. Connection types like the bolted connections located at each floor of the model building would have a realistic coefficient between the two extremes however this is nearly impossible to predict. After comparing experimental to theoretical frequency values which were calculated with a ‘rigid’ assumption, a true column support coefficient of 6.25 was determined to be the most accurate estimation to parallel the first three modes of each model.

Table 4-6 compares the experimental frequencies values with those calculated using the adjusted support coefficient of 6.25.

Table 4-6 Predicted, Adjusted and Experimental Frequency Values

Mode	Frequency (Hz)		
	Predicted	Adjusted	Experimental
1	2.1	1.5	1.5
2	6.3	4.5	4.7
3	10.1	7.3	7.3
4	13.5	9.8	10.8
5	16.4	11.8	13.0
6	18.5	13.3	15.2
7	19.8	14.3	16.8

Adjusting this interstory stiffness and comparing frequency data to experimental response provides an even more accurate numerical model of the physical structural system. The objective of this work was to design, build, and dynamically identify a small-scale structural building model that can be used as a dependable structure with an identifiable behavioral response. It is illustrated by clear frequency response data obtained from multiple dynamic tests that this building model can effectively contribute to structural dynamic tests and be utilized as a baseline structure for future research applications.

3–D Dynamic Building Model

Previous work involving the design and construction of the two-dimensional building not only provided the facility with a new structural model, but also laid a solid foundation for the design and analysis process of any future building to be integrated into the laboratory. Additionally, by significantly expanding the force generating capabilities of the facility by adding the six degree of freedom shake table, larger test structures are able to be introduced to the dynamic testing program, in particular a

larger scale, 3-D building model. Incorporating such a model would not only fully utilize shake table dimensions, force capacities, and multiple degrees of freedom, but provide current and future researchers an opportunity to expand research interests to larger model applications. Similar to the two-dimensional building, a primary objective of this portion of work is to produce a responsive structural model, however an additional goal is to incorporate model versatility allowing for multiple types of experimental set ups to be implemented.

Design

This section details the decision process and provides explanation behind the various design parameters selected such as dimensions, distributed masses, and model adaptability. In some structural dynamic experimental applications, the design of a test model design will be targeted to resemble an existing structure so tests can be applied and direct conclusions can be made relating to that structure. As a general approach, the building model presented in this section was designed to model not a specific structure, but a typical mid-rise steel building by preserving dynamic similitude. This left many design variables to be determined which are mainly based on laboratory space constraints where the structure will be contained, the excitation equipment projected to be used during testing, and overall project expenses for materials and fabrication.

Using past building model design experiences as a guideline, model width (W), height (H), mass (M), fixity (k) and material/cross section geometry (EI) needed to be selected so the dynamic response of the structure could be predicted. To do this, capabilities of the newly installed shake table were used as a starting point to determine the initial direction of design.

Multi DOF motion capabilities influenced the general model design to include three-dimensional member orientation which enabled response motion to occur in both a strong and weak axis. Providing a multi-directional response model would be a progressive step forward from the previous structure which was strictly based on planar motion. Floor pates, supported by multiple rows of columns, were sized based on the 4ft x 4ft shake table platform having a length and width of 2’-9” x 1’-9”. Figure 4-36 illustrates the proposed building and shake table dimensions.

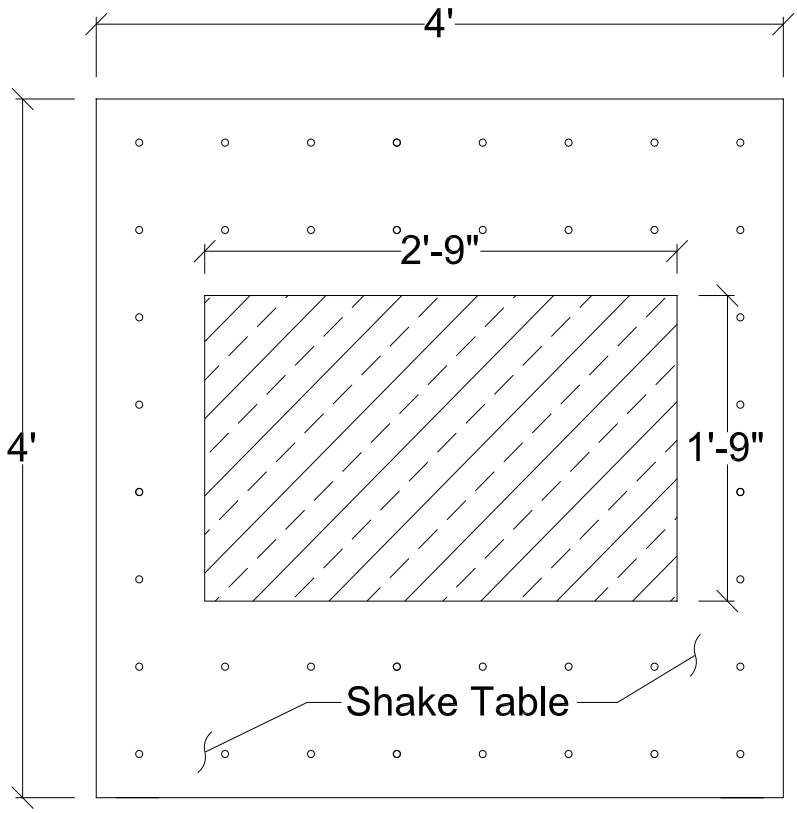


Figure 4-36 Plan view of Building on Shake Table

The plate thickness was determined later as it was influenced by a number of factors that were finalized later in the design process.

With more than twenty feet of clear space available in the facility where the model would be housed, vertical space constraints were not applicable. Investigation

on similar work performed with a steel structural model to be used in shake table applications (Wierschem et al. 2012) helped to narrow this design parameter. Selecting an interstory height of ten inches in combination with a total of six floor levels yielded a total height of approximately 60 inches (with the final dimensions depending on plate thickness). The dimensions chosen provided a workable model for instrumentation, as well as preserving a height to width ratio from previous model designs where successful experimental applications were established.

Aiming for experimental longevity and model durability, structural steel was selected as the primary material. With the project budget being another factor for structural design, fabrication costs were expected to remain lowest if material continuity was maintained. All steel with the exception of the column components was A36 grade and as a preventive measure, columns were designed with high strength steel having an increased yield stress of 100ksi. Increasing the column strength would help prevent the structure from encountering inelastic failure modes and facilitate a wider range of base excitation forces to be generated.

As discussed in the previous section, the difference of a connection's design intentions to what is actually installed on a structure can alter the structural response predicted by preliminary analysis methods. To create a column to floor interface to most accurately resemble a true 'rigid' connection, an assembly of three steel components where a main column element is inserted and welded to a steel flange at each column end would be implemented. This design was based on a previous welded connection detail (Wierschem et al. 2013) where a rigid support condition was intended

and achieved. Welding design and fabrication methods are later explained in the 'Fabrication' section of this chapter.

Similar to the previous building specimen, this model would also contain distributed masses at each floor resulting in a shear-building model. Not only was the magnitude of individual masses adjusted to resemble a typical mid-rise steel structure with natural frequencies from 0-20Hz, but the collective mass of the system was also taken into consideration to confirm the shake table maximum payload of 2,200 lbs was not exceeded. As a conservative approach, mass constraints were not to exceed 75% of payload capacity, resulting in an upper bound of 1650lbs. The referenced building design incorporated the attachment of additional masses at each story; however, due to the steel material of the plates, the option to have the floor plates themselves act as the main mass of the system proved most practical.

Using an iterative dynamic analysis procedure (Appendix A) described previously, design variables from the 6th order shear building model were applied to target the desired mid-rise steel structure frequency range while remaining under the experimentally conservative payload limit. Input parameters were:

- E = 29,000ksi
- H = 9.5 in.
- w = .5 in.
- t=.25in
- k factor = 12 (assumed rigid)
- M = 4.2 lbf

where (E) is steel modulus of elasticity, (h) is interstory height, (w) & (t) is the width and thickness of supporting rectangular columns at each floor, (k factor) is the support coefficient of a rigid connection, and (M) is the mass of each plate. A dynamic analysis was conducted for both strong and weak bending possibilities and provided frequency

response estimations; however, the weak axis bending was used first to finalize building design parameters that satisfied the intentions of the model.

While (E), (H), and (k) variables remained constant in the analysis, mass (M) and column cross section (w and t) progressed through a series of adjustments in order to achieve desired expected natural frequency values. Because these three parameters provided an array of design possibilities, this process became a matter of project practicality including factors from both material expenses and size availability. For example, steel plates proved most economical when ordered in ½" increments. This coupled with a prescribed payload limit of 1650 lbs. provided a starting point for the selection of distributed masses. By preserving perimeter plate dimensions of 2'-9" x 1-9" which was based on shake table platform size, a varying plate thickness of 0.75", 1.0", and 1.5" yielded a total building weight of 1026lb, 1368lb, and 2052lb, respectively. This resulted in the temporary selection of a 1" thick steel plate at each floor level weighing 195 pounds each. Locking this design variable, and using an iterative approach for column dimension inputs, a column width of 0.5" and thickness of 0.25" resulted in the targeted natural frequencies which are displayed in Table 4-7. Preserving these design variables, a second dynamic analysis was performed on the strong axis with increased natural frequency predictions identified below in Table 4-8.

Table 4-7 Predicted Natural Frequencies of 3-D Building Model (Weak Axis)

Mode	Frequency (Hz)
1	2.18
2	6.43
3	10.29
4	13.56
5	16.04
6	17.59

Table 4-8 Predicted Natural Frequencies of 3-D Building (Strong Axis)

Mode	Frequency (Hz)
1	4.37
2	12.85
3	20.59
4	27.13
5	32.09
6	35.19

The increase in modal frequencies between Table 4-7 and Table 4-8 is directly influenced by the change in stiffness based on the column's rotational axis. Predicted resonant frequencies can be compared to experimental results to verify design assumptions and provide a range of frequencies for which the structural model will dramatically respond. The final design is illustrated in Figure 4-37.

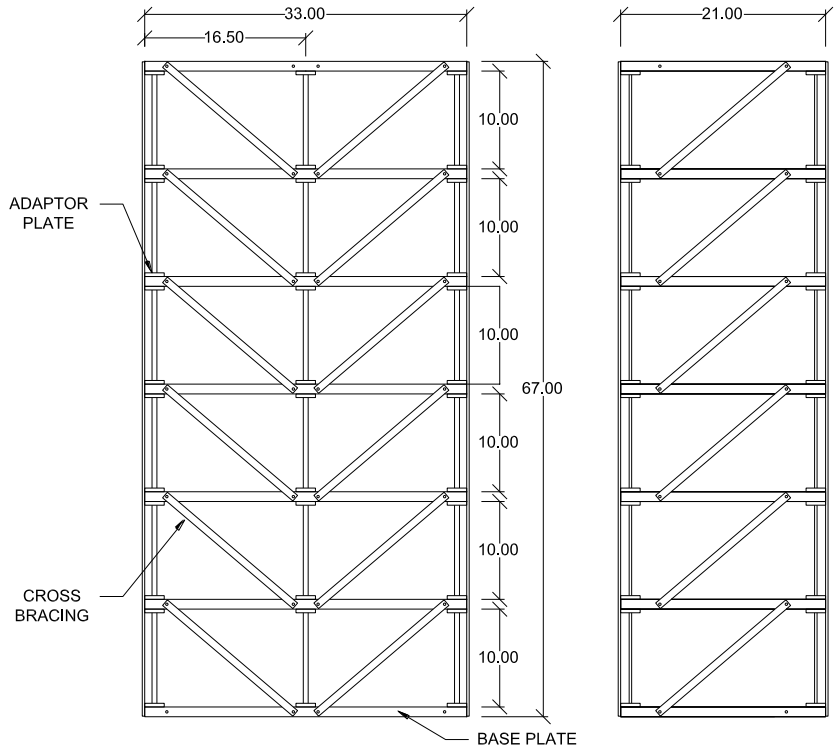


Figure 4-37 Final 3-D Building Design

An additive design component involving interstory cross bracing was implemented as a stability measure during building transportation. Because the targeted frequencies are obtained without these members applied to the structure, anticipated dynamic tests will not include these members since they will significantly add to the lateral stiffness of the model and augment resonant frequency values to an impractical range. Singular cross bracing components however, can be added to test the effects of stiffness discontinuity in a structure.

After all fundamental design checks were complete and confidence in model design was established, shop drawings included in Appendix 1 were created and delivered to the UF Coastal Laboratory where the proper manufacturing equipment is available to fabricate the structure as precise to the design drawings as possible.

Finite Element Model

After design parameters were determined, a finite element model was developed using a general finite element software package (ADINA 8.8). This model was created to not only confirm calculated dynamic characteristics of the test structure, but to identify critical limit states of the model prior to real testing applications.

To most accurately represent the parent building model, two separate element types were implemented. As a result of being subject to both axial compression from gravity and bending moment due to ground motion, columns were represented in the finite element model by beam elements. A series of four-node shell elements were used to model each steel floor plate. It is important to note that plate bending was not anticipated in the 1 inch thick steel plates in comparison to the slim columns so nine-node shell elements were not adopted for this modeling application.

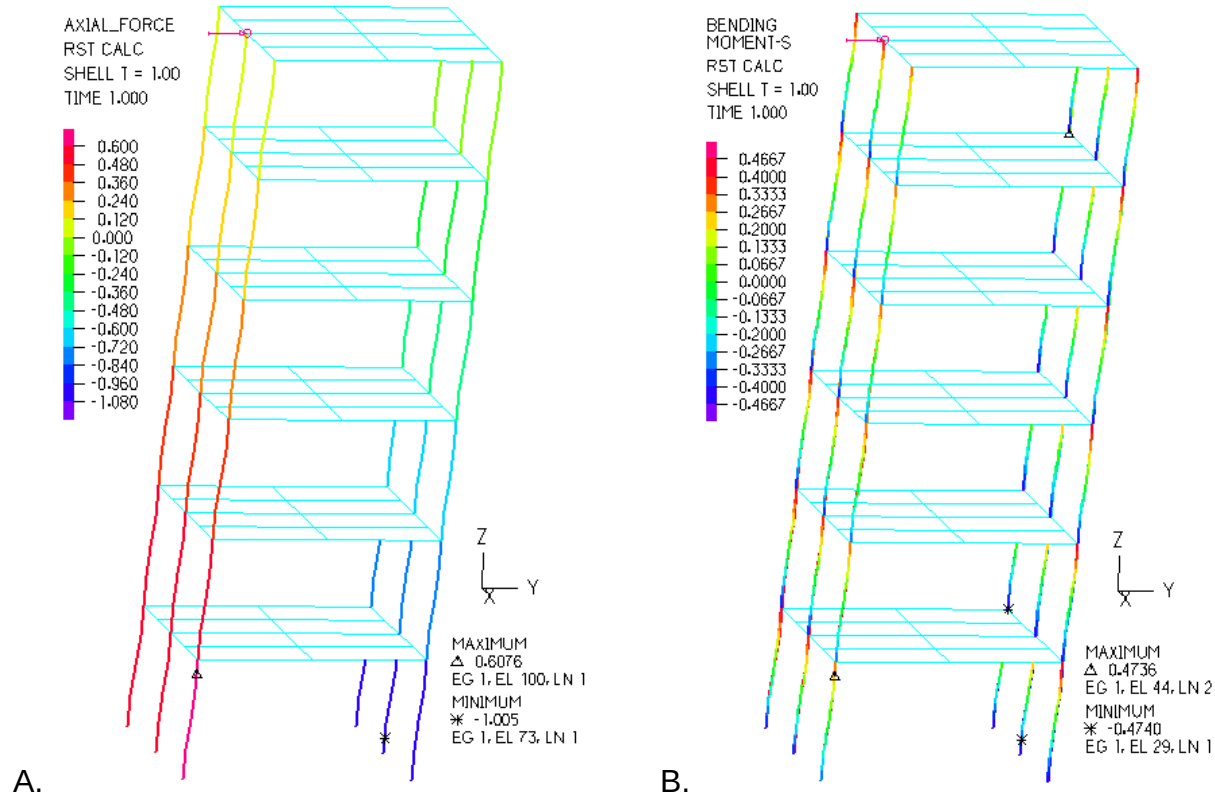
The steel material and member dimensions were represented using a standard modulus of 30,000ksi and column and plate dimensions of 0.25” x 0.5” x 9.5” and 33” x 21” x 1”, respectively. Similar to the previous bridge model, a steel density of 0.00029k/in^3/g was used to account for the total mass of the system.

Global boundary conditions were released allowing for the full three dimensional response of the model. To simulate a fixed connection to the shake table, the bottom nodes were restrained from both translation and rotation in the X, Y, and Z axes.

Relative to plate and building sizes, the selected column cross-section seemed quite small with potential stability concerns. To ensure this structure would not buckle under its own self weight, bucking calculations (Appendix 1) were performed and compared to the FE predictions proving that despite the proportionally small column thickness, each bottom column would only be loaded to 15% of the critical buckling load. This limiting state was quantified using an appropriate rigid fixity assumption based on the welded joint connection.

This structural model is intended to remain a long-lasting structural dynamic research tool in the lab. To limit any permanent deformation during testing, static deflection checks were also performed on the FE model to determine the maximum distance the structure can displace. Due to the bolted connections fixing the base plate of the model to the shake table, inertial lateral forces generated from either ground motion or impact excitation will cause the structure to respond similar to a cantilever beam. As a result, highest stress will occur in the base columns where translation and rotation are both restrained for bottom nodes. Figure 4-38 presents a visual of the FE

model subject to an 830 pound static load corresponding to an effective yielding stress of the bottom columns.

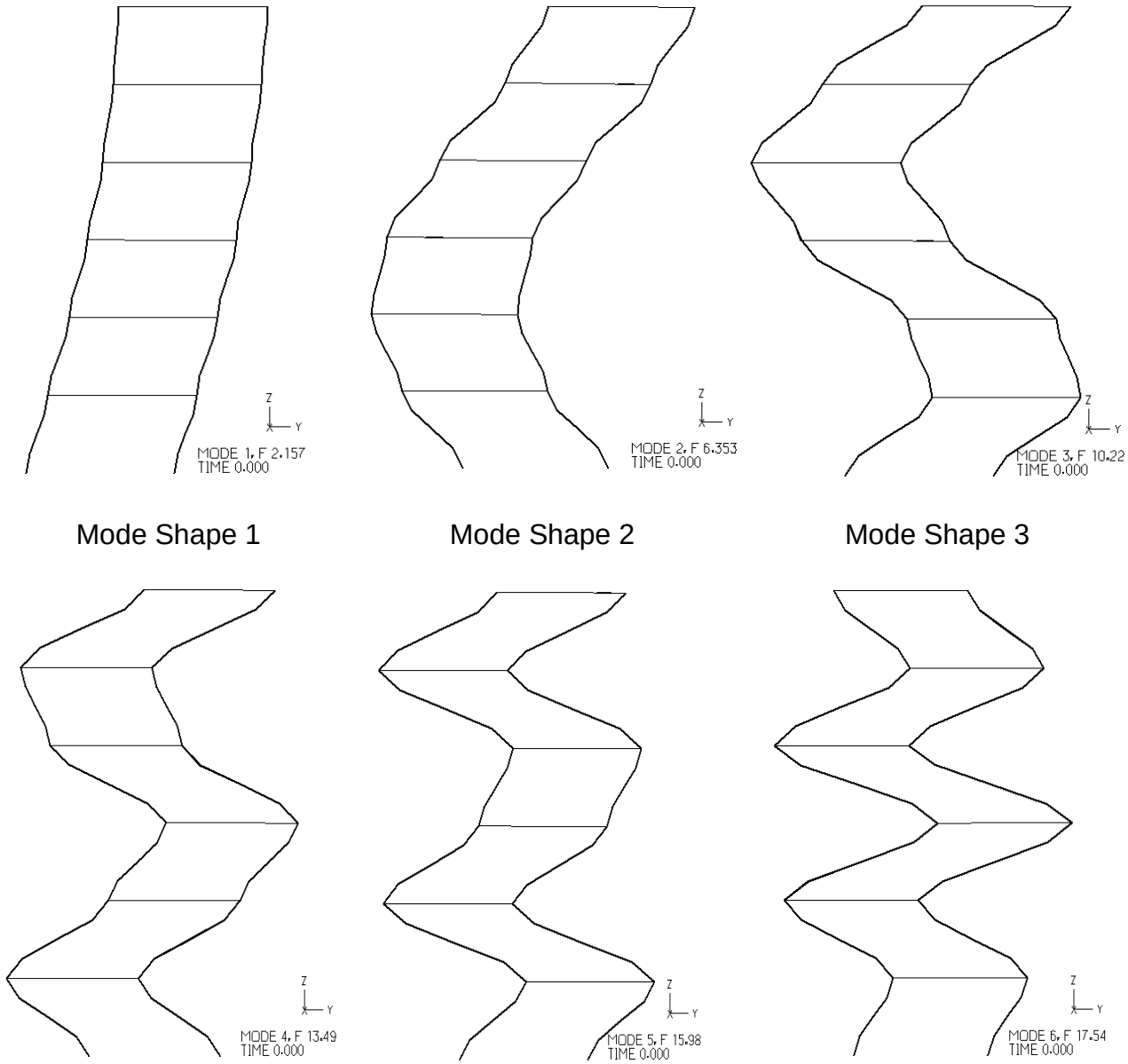


Axial Forces Bending Moments

Figure 4-38 Axial Forces and Bending Moments

Incorporating both axial force and bending stress components, effective stresses were calculated for critical elements. Top story displacement of 3 in resulted in base columns reaching their 100 ksi yielding stress. Using this maximum displacement value as a limiting constraint for dynamic testing, preventive measures will be instilled to ensure generated forces will not invoke a top level displacement of this magnitude.

Using the modal analysis feature of the finite element program, previous natural frequency calculations listed in Table 4-7 and Table 4-8 were confidently verified. Illustrated in Figure 4-39 are the six mode shapes in the limiting weak axis of motion.



Mode Shape 1 Mode Shape 2 Mode Shape 3

Mode Shape 4 Mode Shape 5 Mode Shape 6

Figure 4-39 FEM Predicted Modes for the 3-D Building Model

The development of a finite element model is a valuable component to the structural dynamic research done in this lab. With a working and accurate computer model, potential experiments can be simulated and preliminary results can be anticipated and compared. This will prevent any misuse of the structural model and shake table combination. After the verification of the model's design and dynamic response via a

FE model, fabrication, detailed in the next section, was the subsequent phase in the development of this structural model.

Fabrication

In the previous structural models identified in this work, such as the two dimensional building model, “in house” fabrication by the structural designers was possible due to the minimal costs associated with material and the smaller scale of the project. In the case of the three dimensional building model, where the scale and level of detail is increased, it was decided to seek a third party fabricator to ensure quality and precision of the various components required in the model design. After a cost analysis of different fabrication avenues, the University of Florida Coastal Engineering Laboratory had the capabilities, personnel, and availability to perform a project of this magnitude. As a result, this section will provide a general outline of the fabrication phases with detailed plan sets and photos included in Appendix A.

With multiple steel components in the building design, a total of four different steel sizes were delivered for assembly. Floor plates were shipped in the design dimensions of 33” x 21” x 1” with accuracy tolerance of +/- 1/16”. To accurately place the fastener holes in the proper location, a vertical drill and mag drill was used to cut the face through holes and edge holes, respectively. Cross bracing, delivered in precut sizes that matched the dimensioned required by design, were drilled with ¼” through holes to fasten to the edge of each floor.

Although floor plates weight approximately 200 pounds each, the columns posed to be the most challenging fabrication component in the structure. To preserve a rigid connection, the column – flange interface was designed as a welded joint; Figure 4-40provides a sketch of the designed column.

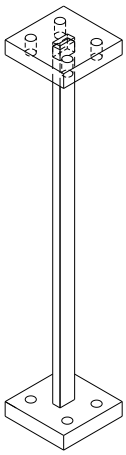


Figure 4-40 Designed Column

Individual columns were cut every ½” from a 10” wide x ¼” thick sheet of A514 high strength steel. Organized in Figure 4-41 are photos corresponding to the general process of the column assembly.

Figure 4-41 Column Fabrication Process

The process illustrated Figure 4-41 is explained here. The ends of each column face are grinded down about 1/32” to allow for a snug fit into the precut 2” x 2” x 3/8” steel plates acting as the top and bottom column flanges. These plates not only provide an element for column connection, but also are large enough to provide bolted connection points to fix the assembled column to floors above and below. The three component set is then fit into a steel frame to allow for accurate alignment while welding. The full penetration weld layers (full details in Appendix A) are applied until the void is eliminated and the final column assembly is then smoothed for a final product. Figure 4-42 shows an assembled column – floor plate interface with cross bracing attached.

Figure 4-42 Joint Interface

Using bolted connections to connect the welded column flanges to the floor plates, each node closely resembles a rigid connection. Subsequent dynamic experiments are used to verify this connectivity assumption. Figure 4-43 shows the erected building model after the final stages of assembly.

Figure 4-43 Erected 3-D Building Model

By using the appropriate fabrication methods and equipment, this three-dimensional building model will not only be used in various structural dynamic experiment applications but should prove to be a valuable research tool for future research projects.

Dynamic Testing and Results

After the construction of a building model, the next step is to apply the test specimen to dynamic experiments to quantify its dynamic response. Similar to the previous building, test results can be used to verify or rectify a design assumption such as a boundary condition or joint stiffness to accurately represent physical experimental model.

As shown in Figure 4-43 the steel structure is separated by individual components consisting of a plate at each floor and six connecting columns above and below the intermediate stories. By utilizing this configuration type, multiple structures could be tested ranging from a three to a fully erected six story building. Using previous dynamic testing procedures from past structural models as a basis, both impact and forced vibration excitation methods were applied to the various models with subsequent modal analysis to characterize each structure and compare to previous calculations. Although this test specimen provides the ability to test multiple building models with altered heights, this section will elaborate on the instrumentation and testing methods of the six story model followed by the supporting results from other building compositions.

Instrumentation. The building has the potential to respond in each of the strong, weak, and torsional axes. In prior building instrumentation only one chord of sensors was needed to monitor the planer response motion of the model. The three-dimensional response of the steel building in combination with the limited number of sensors available in the laboratory presented a more detailed instrumentation process than the previous applications explained in this work.

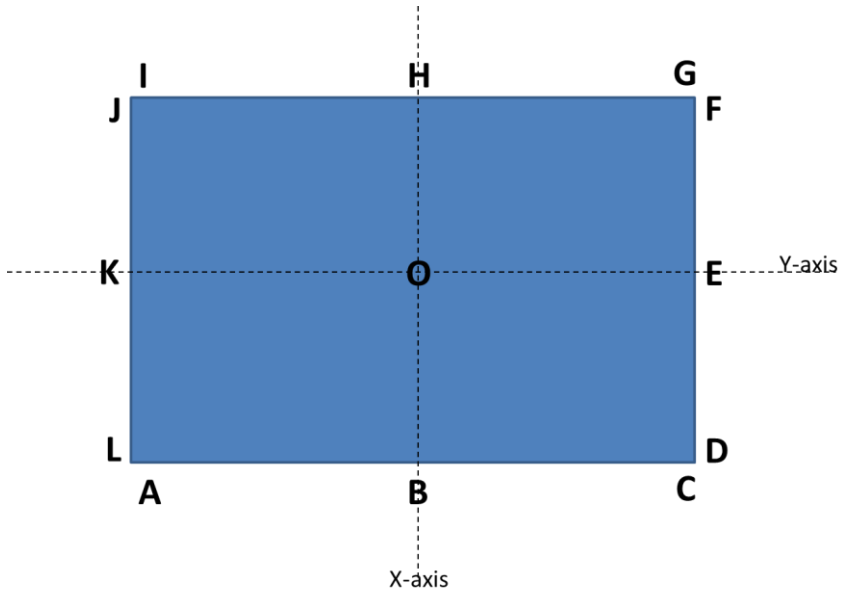


Figure 4-44 3-D Building Labeling Convention

Figure 4-44 illustrates the selected labeling configuration of each floor plate. Based on the supporting column orientations, the X and Y axes identified in Figure 4-44 are associated with the strong and weak axes, respectively. To monitor vibration response of the structure, the same uniaxial accelerometers are used as in previous testing applications. This required each corner to have two possible sensor positions to capture dominant vibrations in either the X or Y axis. When selecting sensor positions it is most common to select locations associated with anticipated maximum limit states or vibration amplitudes. For example, accelerometers are used to capture vibrational response and should be placed in areas that experience maximum relative displacement (corners of each plate). If member strength is a concern, strain transducers should be placed where maximum stresses are anticipated to occur (bottom floor columns). In the case of the building, corner nodes are the area of interest due to the larger difference in vibration between the intermediate stories. Figure 4-45 identifies the sensor layout of the 3-D building model.

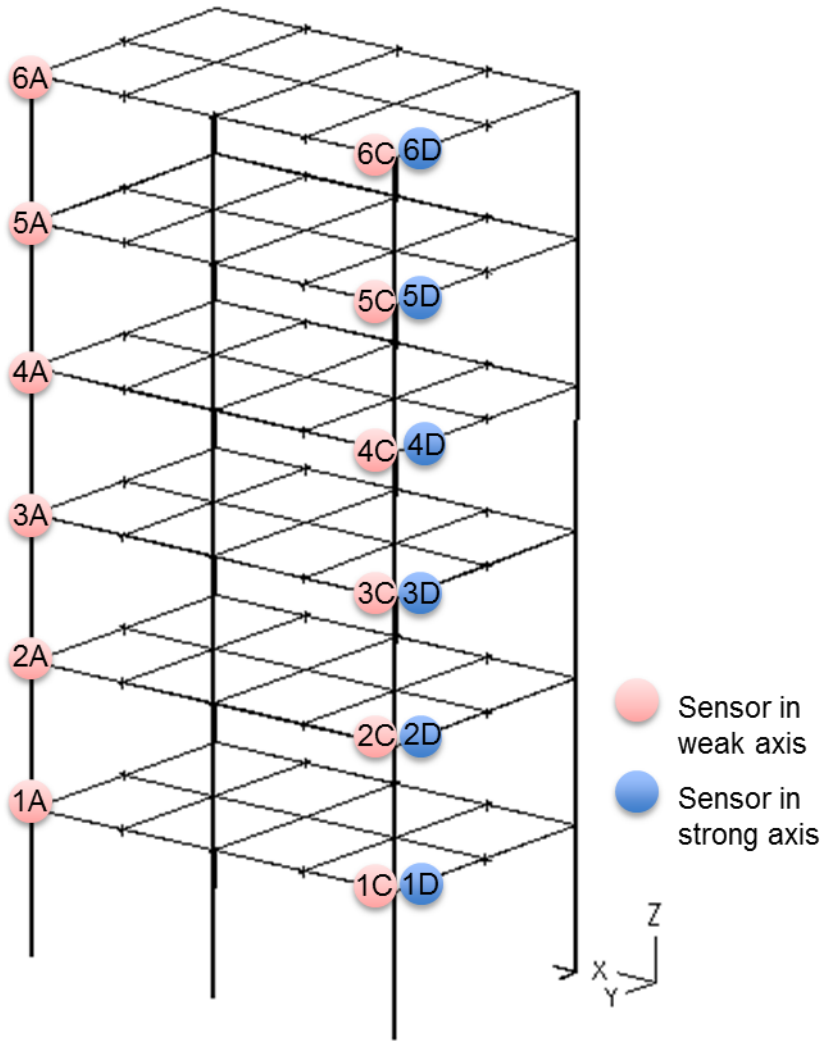


Figure 4-45 3-D Building Sensor Layout

Shown above in Figure 4-45 with reference to the labeling convention in Figure 4-44, sensors were placed on the A, C and D column ends at each floor level. This sensor arrangement provided the most efficient use of the 20 channels available by the data acquisition system. By applying sensors on the A and C columns in particular, weak axis bending response can be recorded. Additionally, the single line of sensors at chord D captures strong axis vibrations. Torsional modes are quantified by comparing the difference in phase between the A and C columns. If these sensor groups are out of phase from one another, torsion is considered to be the dominating mode and can then

be quantified accordingly. Thus, using this sensor distribution layout provides the most effective orientation to monitor the global response of the structure and provide sufficient data to perform modal analysis calculations.

Impact testing. To quantify dynamic response, impact tests by use of an impact hammer were performed first as a fast and effective dynamic testing method. Tests were conducted during the building assembly process allowing for impact testing and dynamic characterization of a three, four, and five story building model. To monitor vibrational response of all three dimensions multiple impact locations were needed. Figure 4-46 identifies the location and direction of impact for each of the four building models.

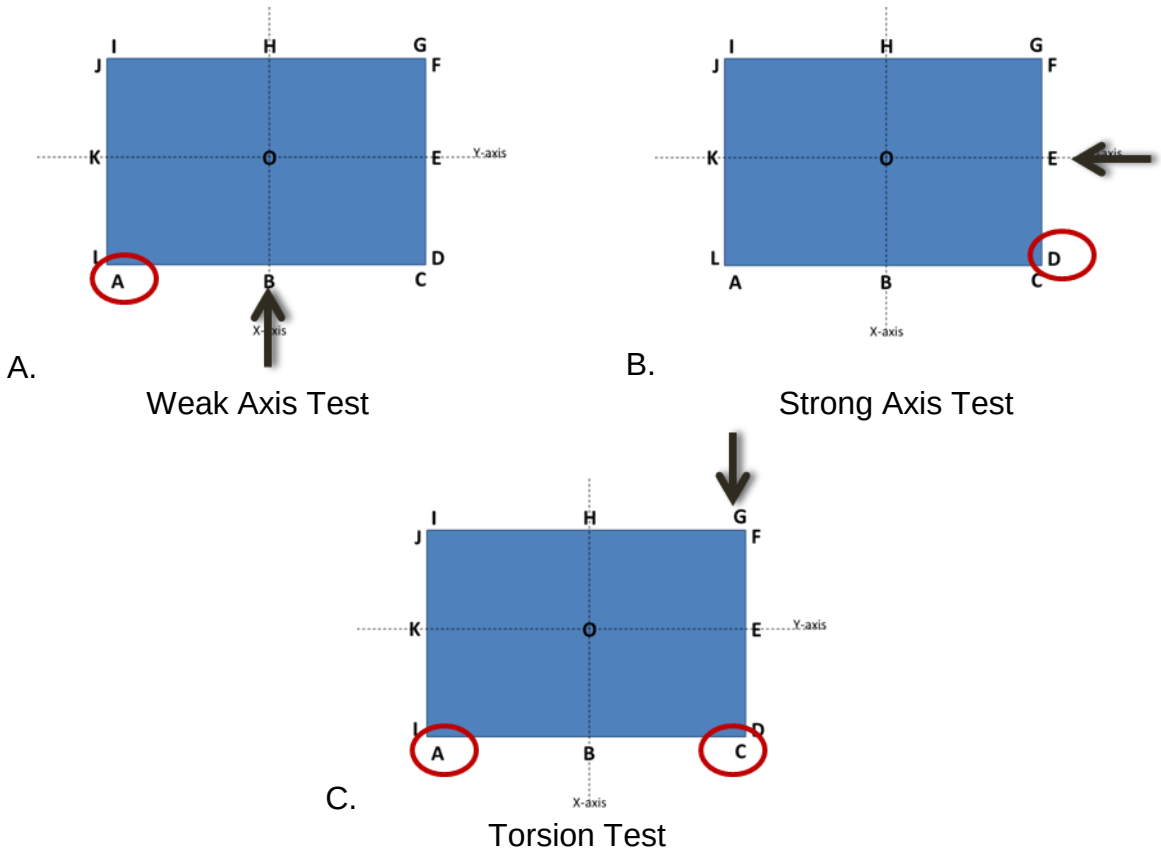


Figure 4-46 Impact Test Set Up

Impact to the top plate of each building was applied at Node B to isolate weak axis response and quantify associate modal frequencies in this direction. Plotted in Figure 4-47 is the frequency response of sensors columns A from impact to Node B at the top floor of the fully erected six story building model.

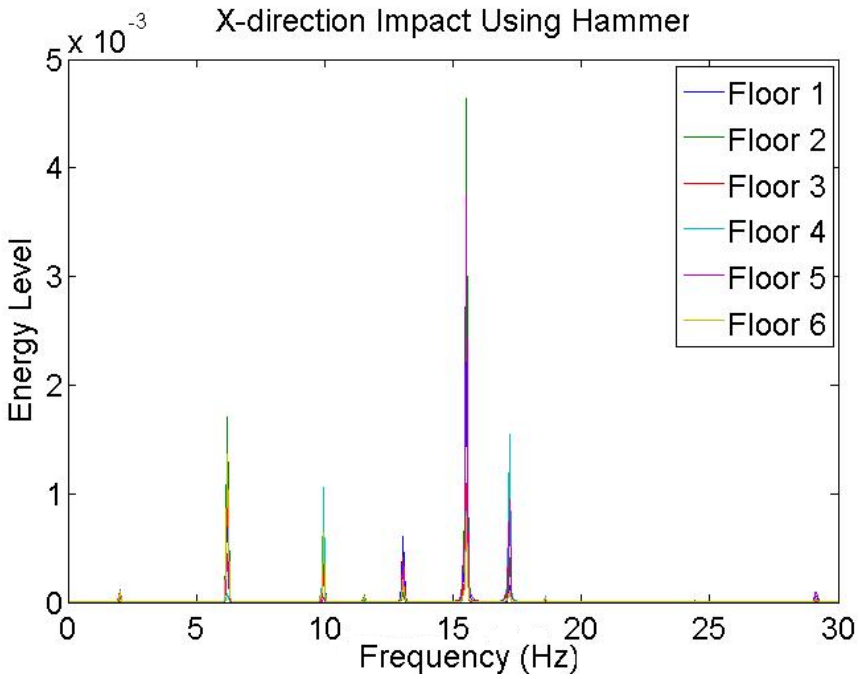


Figure 4-47 Weak Axis Frequency Response from Impact

Visually clean peaks of energy are associated with dominating weak axis vibration response. As a result, confident weak axis frequency values are selected for each of the six modal frequencies as 2.18, 6.43, 10.29, 13.56, 16.04, and 17.59 Hz, respectively.

A similar approach was preserved for a second round of tests where impact was applied to Node E with the intention of the strong axis to be the dominating bending mode. A similar trend of results is illustrated in the frequency domain for strong axis response of sensors at Node D as a result of impact tests; this is displayed Figure 4-48.

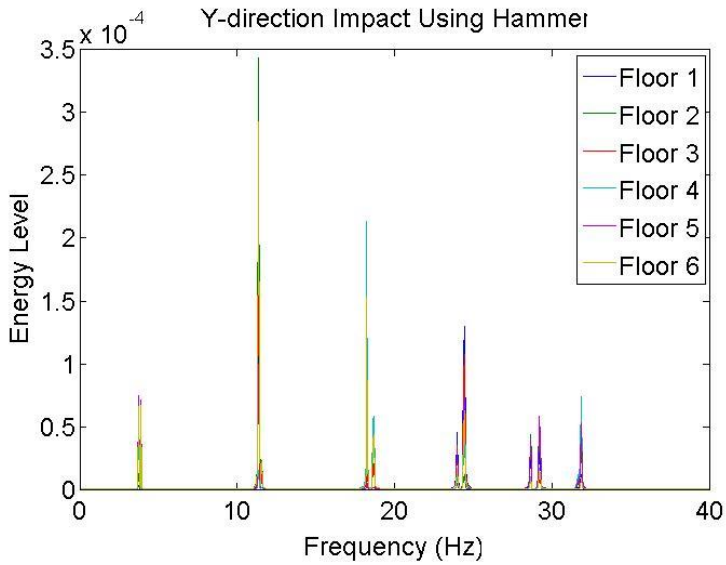


Figure 4-48 Strong Axis Frequency Response from Impact

When comparing weak and strong axis response, there is an expected increase in frequency values as a result of the increase in stiffness along the strong axis. After further review of frequency response data in Figure 4-47 and Figure 4-48, smaller peaks can be observed between the larger energy peaks. This is due to a portion of the input energy distributed to torsion response modes of vibration

In the case of this 3-D building where torsion components may dominate the response of applied excitation, it is important to first identify torsion frequency peaks, particularly when implementing the “peak picking” method for system identification. This allows for a subsequent selection of both strong and weak frequency peaks. To isolate torsion response, impact was applied to a nonconcentric nodal location; Node G. This forced a combined response of the strong and weak axis and allowed for the calculation of torsion modes by analyzing the phase of sensors location at A and C. Plotted in Figure 4-49 is the frequency response data of the 6-story building with data configured to separate a torsional mode from a bending mode.

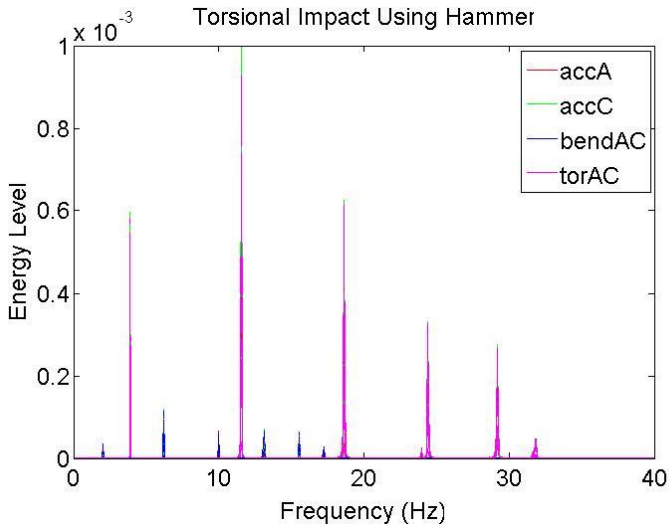


Figure 4-49 Torsional Frequency Response from Impact

By isolating the sensors located on area A and C of each floor and analyzing the phase data between each, the torsion frequencies can be both visualized and quantified. In Figure 4-49, high energy peaks are associated with torsion modes and low energy peaks are identified as strong or weak bending modes of vibration. Resulting natural frequency values are collected in Table 4-9 for the six story building configuration after the impact tests were applied to all dimensions of the structure.

Table 4-9 6-Story Building Natural Frequencies

Mode	Frequency (Hz)		
	Weak Axis	Strong Axis	Torsion
1	2.18	3.72	3.9
2	6.43	11.35	11.54
3	10.29	18.85	18.62
4	13.56	23.99	24.41
5	16.04	28.63	29.17
6	17.59	31.68	31.86

Preliminary design parameters for the structural model were selected so natural frequency ranged from 0-20Hz. As shown in Table 4-9 the intended dynamic response was achieved in the actual experimental model. As mentioned previously, this model

presented an opportunity to conduct impact tests on multiple buildings each with different heights and the dynamic testing procedure implemented on the fully erected structure was preserved from these previous tests. Presented in Figure 4-50 are the combined natural frequency results from the three, four, and five story building models.

3-Story Building			
Mode	**Frequency (Hz)**		
	Weak	Strong	Torsion
1	3.85	7.02	7.45
2	11.1	20.08	21.12
3	16	28.99	30.46
-	-	-	-

4-Story Building			
Mode	**Frequency (Hz)**		
	Weak	Strong	Torsion
1	2.91	5.43	5.74
2	8.39	16.05	16.54
3	12.85	24.72	25.39
4	15.77	30.33	31.37

5-Story Building			
Mode	**Frequency (Hz)**		
	Weak	Strong	Torsion
1	2.89	4.46	4.64
2	7.4	13.37	13.61
3	11.7	21.3	21.73
4	15	27.34	27.77
5	17.2	31.19	31.68

Figure 4-50 Three, Four, and Five Story Building Experimental Results

As the number of stories increase from three to five, it can be seen that natural frequency values decrease in magnitude. For example, weak axis mode 1 frequency values steadily decline from to 3.85, 2.91, and 2.89Hz for the three, four, and five story buildings respectively. Similar trends can be found in the strong and torsion frequency values of each mode shape. This can be attributed to the increase in overall mass of the structure which has a direct influence on the inherent frequency.

Ambient vibration testing. Past excitation equipment and model combinations consisted of the uniaxial shake table for the 2-D building model and the long stroke vertical shaker for the steel bridge truss. Following the natural progression of past dynamic testing procedures, the 6 DOF shake table could now be implemented as the excitation source for random vibration of the building. As mentioned in the shake table section of this work, the table payload capacity is 2,200lbs. Although the fully erected building model is roughly 1,500lbs, a conservative approach was taken for the first round of tests of this new structure. Because this was the first structure to be subject to motion by the shake table, it was decided that initially testing half of the model was a safe and logical test strategy when integrating new excitation equipment into the lab. After three-story building shake table tests were complete and confidence was gained in testing procedure and system demand, the same ground excitation was applied to the fully erected six-story structure. Figure 4-51 shows the building and shake table combination just before applying base excitation via the newly installed shake table system.

Figure 4-51 Six Story Building Fixed on Shake Table Platform

Eight ½ “bolts were used to fasten the building model to the shake table platform. This provided a fixed connection to the table and ensured minimal damping effects from the ground to floor plate interface. Internal LVDT sensors in each actuator were used to collect reference ground motion displacements. With known input energy and collected response vibrations a full system identification can be made available for future work with the structure.

Referring to previous impact test results, a range of physical frequencies for which the building will response could be drawn from to set an upper and lower frequency limit on an excitation signal. A band limited white noise signal with frequency range from 0-20Hz was applied as a ground excitation in the X-axis to isolate weak axis vibration response. Figure 4-52 shows the applied displacement time history record of for the ambient vibration test.

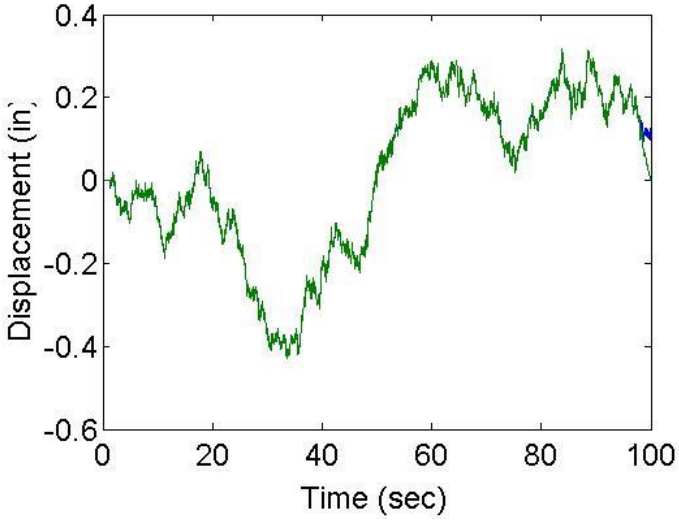


Figure 4-52 X-Axis Excitation on 6-Story Building Model

With displacement RMS amplitude of 0.25 inches this excitation was applied for 100 second duration and provided equal energy content at all frequency values in specified range with the intention to supply adequate energy for each of six modal frequencies of the building to be excited. Figure 4-53 illustrates the building frequency response when subject to ground motion along the X-axis.

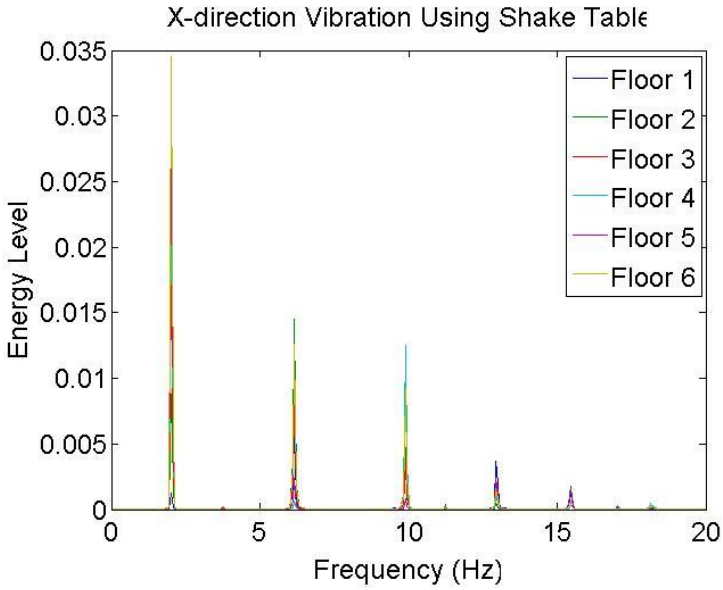


Figure 4-53 Frequency Response of 3-D Building

After appropriate post processing of data and applying modal analysis, calculated natural frequencies were consistent with those found from impact tests. Figure 4-54 shows resulting mode shapes in the X-direction from the building response data when exposed to shake table vibration as it compares to the calculated mode shapes from preliminary analysis.

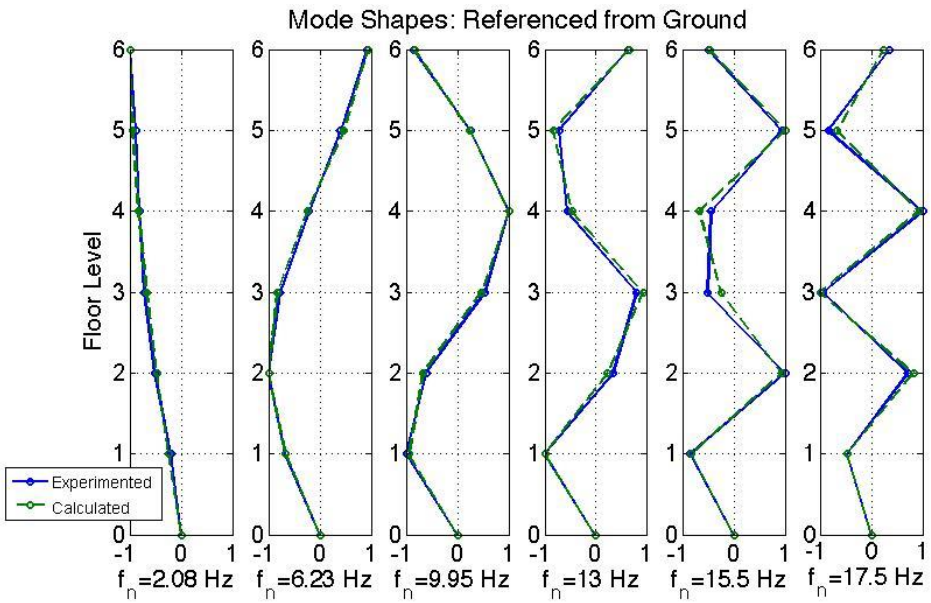


Figure 4-54 6-Story Building X-Axis Mode Shapes

Falling between the intended 0-20Hz frequency range, the calculated mode shapes are plotted against the experimental modes shapes to illustrate the minimal dynamic variation between the design and fabrication phases.

Predicted vs. Experimental. After the final tests were conducted and accurate system identification was obtained, the calculated natural frequency values from the lumped mass modal analysis were compared to experimental findings. Presented in Table 4-10 are compared natural frequency values in both the strong and weak dimensions.

Table 4-10 Predicted vs. Experimental Natural Frequencies

Mode	Frequency (Hz)			
	Weak Axis		Strong Axis	
	Predicted	Experimental	Predicted	Experimental
1	2.18	2.08	4.37	3.72
2	6.43	6.23	12.85	11.35
3	10.29	9.95	20.59	18.85
4	13.56	13.00	27.13	23.99
5	16.04	15.50	32.09	28.63
6	17.59	17.50	36.19	31.68

After analyzing predicted versus experimental natural frequency data, design assumptions are confidently confirmed. There is larger frequency differential for the strong axis results when compared to the weak axis. This may be attributed to a few variables that can arise in any fabrication process of a structural model, in particular the column flange connections. The welded joint connection on the top and bottom of each column detailed earlier in this section may have minor internal inconsistencies as the human element is introduced.

After initially characterizing the structure by quantifying natural frequencies, it would be most appropriate to pursue other system identification techniques to validate current test results and to also quantify other inherent dynamic properties of the structure. A damping ratio of 0.05 was used in preliminary design and analysis however with input and output response data now available, a natural damping ratio can be calculated for use in future structural dynamic applications.

Presented in this chapter were three individual structural models that each had a unique component and particular scope of work and each subject to multiple phases

including assembly, design, fabrication and construction. In-depth documentation and explanation of these project phases have developed this work into a valuable resource for any future researcher looking to pursue a similar area of structural dynamic research.

CHAPTER 5
CONCLUSIONS AND RECCOMMENDATIONS

The overall goal of this research was to plan, design, and implement a fully functional research laboratory that provides a broad array of structural dynamic testing capabilities. The final outcome is a laboratory facility with three calibrated structural models representing a number of structural forms with the flexibility for modification to support a variety of experimental campaigns. Additionally, this research resulted in three dynamic excitation instruments with a range of capabilities and configurations to enable testing of many sizes and forms of structural models, components, and sensors.

The scope of work presented in this document involves a variety of projects, all aimed to contribute to this overall goal. The main components of this research have been separated into two distinct chapters according to their functional role in the operation of the laboratory. Each project is thoroughly explained to provide a body of reference for both the design basis and research intentions of the facility.

The excitation equipment employed in the laboratory consists of three instruments that are identified as either small or larger scale. Due to minimal installation and setup demand, the small scale devices served as an expedited approach to apply excitation for initial research interests such as sensor development. As identified in Chapter 3, these devices consist of a uniaxial shake table and a long stroke shaker.

On a larger scale, a 6 DOF shake table was integrated into the facility to increase the structural dynamic experimental potential and address new research interests. Not only was this piece of equipment implemented on a larger geometric scale than previous models, but it involved a multi-dimensional project management component due to a variety of project phases. As a requirement of this work, explanation on the

assembly, installation, and operation of the shake table system is documented, indicating the necessary planning and installation required for the successful completion of a project of this magnitude. Upon completion of these project phases, a calibrated and functional shake table system is available for appropriate research applications, including an operational procedure documented in a comprehensive, laboratory-specific User's Manual. Before any significant excitation approaching the specified performance values is applied by the table, it is recommended that a full shake table characterization be performed. This characterization will allow for true performance limits to be recorded since these limits may differ slightly from the specified values due to the variability in the hydraulic system.

As structural models were added to the laboratory inventory, all three pieces of excitation equipment presented in this research were used in multiple instances to apply energy into these structures. These dynamic experiments enabled modal identification of each of the structural models. While the design and application intention of all of the excitation equipment is to be a useful tool for general dynamic research, the newly installed shake table system also enables experimental work specific to the field of earthquake engineering.

The structural models introduced in this research each have a unique purpose and research intention but all provide a valuable tool directed toward the overall field of structural dynamic research.

The prefabricated, predesigned three-dimensional bridge model was promptly adopted into the facility with an initial project phase of construction and assembly. With the proper means and precision assembly methods outlined in this work, a workable

bridge model was integrated into the laboratory testing program within a few months. When compared to the dynamic response of a full-scale bridge, the assembled structure possessed an overly low horizontal stiffness and an overly high vertical stiffness. As a result, two modifications were made to the original bridge structure. To address the overly flexible horizontal component, horizontal cross bracing along the bottom bridge deck was added to provide lateral force resistance. As illustrated by experimental modal analysis, this bracing significantly increased horizontal frequencies creating a stiff bridge deck with a more flexible top layer beam system which is a closer representation of an actual bridge. Extending the original bridge span by 2.5ft (one extra bay) the originally high vertical stiffness was reduced but is still demonstrating high vertical frequency response.

One critical component to the bridge portion of this work was the creation of finite element (FE) bridge model. Using predicted modal frequencies as a reference, a functional relationship was established with experimental results in both the horizontal and vertical directions. These relationships were implemented to estimate ranges of frequency response associated with a modal frequency. By witnessing the benefits this research tool, a new procedure to include a FE model for future structural model testing was suggested and implemented.

One challenge encountered during the various stages of bridge testing was the inability to impose adequate energy to excite the higher frequency modes of the bridge. This was the case particularly for the vertical motion component. As discussed, the long stroke shaker applied to the bridge for vertical excitation cannot impose adequate energy at the high range of frequencies in the vertical direction. It is suggested that

future work conducted with this bridge model utilize an alternate shaker system with adequate high frequency energy generating capabilities to ensure reliable diagnosis of the vertical dynamic response.

The two-dimensional, seven-story building model included in the testing program encompassed project stages associated with design, construction, and structural dynamic testing. The purpose of this small scale model was to introduce a responsive structure for which structural dynamic response can be easily and confidently attained. The design methods, testing procedure and subsequent modal analysis pertaining to this structure was to serve as a reference for the three-dimensional building model intended to be used in combination with the newly installed 6 DOF shake table. After applying both impact and random white noise base excitation, the structural dynamic characteristics were obtained, recorded and compared to preliminary analysis to confirm the calculations were properly performed. This model can be adopted in many future research applications as a benchmark structure with quantified dynamic response and can also be applied for general education on structural dynamics.

The three-dimensional, six story building model implemented for this facility had similar project stages as the two-dimensional building involving design, assistance in construction, and dynamic testing to quantify dynamic response. The multiple project phases of this model drew from the various experiences with previous models. An FE approach was adopted from the previous bridge project after observing the benefit of a incorporating a working FE model to a project. Additionally, basic building design principles were taken from the previous design from the two-dimensional building. Both static and modal analyses were applied to the FE model to quantify various limit states

and verify preliminary design prior to the construction of the building. After fabrication of beam and column elements and construction of the building model system, strong, weak, and torsional impact tests were conducted to obtain the global dynamic response in all dimensions. The 6 DOF shake table was then used to impose a random white noise base excitation to the six story model and verify the impact test results. Future work with this structure can include a variety of testing applications. Model versatility allows for the ability to interchange elements that can vary in cross section or material type, add concentric or non-concentric masses at each floor, or to apply cross bracing on selective sides of the building. Additional experimental possibilities can be developed with the excitation capabilities of the shake table and can include motions on a multi-axial plane as well as seismic simulations. A future recommendation for this model involves the application of alternate system identification methods in an effort to provide a confidence level to a given mode shape or to quantify other dynamic parameters such as inherent damping ratios.

All three models developed in this work were created to obtain structural dynamic characteristics to be used in future research applications. The development of these models followed by the testing procedures highlighted in this work provided the components necessary to accomplish this research goal. By incorporating three different excitation instruments and structural dynamic models into a testing program, the facility discussed in this work has the appropriate resources to provide current and future researchers a platform to conduct a wide variety of structural dynamic research experiments.

APPENDIX A
DESIGN DRAWINGS

Design drawings can be found in the University of Florida Institutional Repository at: http://ufdc.ufl.edu/IR00003553/00001

APPENDIX B
CALCULATIONS

Calculations can be found in the University of Florida Institutional Repository at:
http://ufdc.ufl.edu/IR00003553/00001

APPENDIX C
USERS MANUALS

Users Manuals can be found in the University of Florida Institutional Repository at: http://ufdc.ufl.edu/IR00003553/00001

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BIOGRAPHICAL SKETCH

Justin Roger Martinez was born in Norwich, Connecticut in November of 1988 and was raised in San Antonio, Texas where he developed a passion for the sport of baseball. To prolong his baseball career, Justin attended Laredo Community College where he received an Associate of Science degree. This was followed by two more years as a student-athlete at the University of Texas at San Antonio where he obtained a Bachelor of Science in Civil Engineering. With a graduate research assistantship opportunity available, Justin pursued his interest in the structural engineering discipline and obtained a Master of Science in Civil Engineering from the University of Florida.