AN INVESTIGATION OF HUMAN INDUCED VIBRATIONS ON CLEMSON MEMORIAL STADIUM

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ABSTRACT

Human induced stadium vibrations have become of increasing concern to design engineers and facility managers as more energetic crowds and increasingly efficient structures have increased the likelihood of serviceability issues. The “in-time” bouncing of spectators due to music has caused one such issue in the south upper deck of Clemson University’s Memorial Stadium. To gain an understanding of the severity of vibrations in Memorial Stadium, a combination of experimental and analytical modeling is utilized. Data collected by an array of accelerometers installed on a main post tensioned precast cantilever girder provides a season and a half of response histories in various states of occupancy and excitation.

Utilizing data obtained due to ambient vibrations, the Enhanced Frequency Domain Decomposition (EFDD) methodology is used to extract modal properties including frequencies, mode shapes, and modal damping at various levels of occupancy. A finite element model of the girder, initially created utilizing structural drawings and design level properties for an empty stadium, is calibrated using the dynamic characteristics of the empty stadium. This process is carried out considering the first three modes. The calibrated model can be used to analyze various crowd loading scenarios including passive, active, and harmonic loadings with known responses and unknown responses.
Comparing the results of the FE model to relevant data sets shows the EFDD method in combination with ambient vibration data to be an effective means of analyzing a large structure that would have otherwise been difficult to analyze by traditional forced vibration techniques. The calibrated model provides a benchmark for future health monitoring or performance studies.
DEDICATION

I dedicate this thesis to my parents, Scott and Gayle Morris, my brother, Jeff, and rest of my Clemson family for their unending guidance and support.
ACKNOWLEDGEMENTS

I would first like to thank my professors, Dr. Bryant Nielson, Dr. Scott Schiff, and Prof. Stephen Csernak. This project would not have been possible without their help and guidance. I would like to thank the Clemson University Athletic Department for funding this project. A special thanks is in order to Gary Wade of Athletic Facilities, he truly had all the keys to my success. I want to thank John Elsea and Daniel Metz for all of their help throughout the project, most notably for their assistance in installing the instrumentation in the stadium. Additional thanks goes to Dr. Samit Ray Chaudhuri of the University of California, Irvine, for his assistance in the implementation of the Frequency Domain Decomposition Method.
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CHAPTER ONE
INTRODUCTION

With the increased availability of computing power, more and more complex models are being created to analyze structures. Although the ability to analyze structures more accurately is available, the assumptions made to generate these models restrict the overall accuracy that can be obtained. In an effort to test the assumptions needed to compute a model, the dynamic characteristics of the in-service structure can be used to update the model such that the model more accurately resembles the physical structure. The increased accuracy of the model is warranted in situations in which analysis, rather than design, is being conducted. In design, assumptions are made on the side of caution, ensuring the structure will have adequate strength and stiffness. For analysis, an emphasis is placed on accuracy rather than caution. An accurate model has many uses in the field of structural engineering including:

- Use as a baseline for structural health monitoring and damage detection.
- A starting point for rehabilitation investigations.
- Extreme event investigation.
- Investigations into serviceability issues.

It is the last of these uses that is the emphasis of this research.
1.1: Project Description

The increased use of more efficient structural systems and a greater demand for obstruction-free sight lines, have led stadia to become increasingly susceptible to vibration based serviceability issues. The combination of lightweight and flexible structural systems with increasingly lively crowds has seen many modern stadia facing concerns about human-induced vibrations. Frank Howard Memorial Stadium on the campus of Clemson University is no exception. Expanded in 1980 to include a south upper deck, Clemson Stadium went a quarter century with few concerns about the integrity of the expansion. That changed in 2006 when letters of concern were received from spectators experiencing unpleasant levels of vibrations in the south upper deck. The vibrations were felt during the playing of Zombie Nation’s song, *Kernkraft 400*. The playing of this song stimulates the in-time jumping of a portion of the crowd, causing the stadium to vibrate in response.

The overall objective of this research is to produce a finite element model (FEM) that can be used to investigate the phenomena. Through stadium monitoring during in-service and empty situations, the dynamic characteristics of the stadium can be identified using system identification techniques. Using the identified dynamic properties of the stadium, an initial finite element model (FEM) of a portion of the stadium can be updated using model updating techniques. The goal of the model updating is to match the dynamic properties of the FEM to the dynamic properties of physical structure by varying
structural parameters. The updated model can then be verified by comparing analytical results generated by the model with results measured during in-service monitoring of the stadium. The end product is a model that will allow the user to determine the response of the stadium to changes in activity level of the crowd, crowd configuration, or forcing frequency. Furthermore, the updated model can be used as the basis for investigating any improvements that have been deemed necessary to counteract the perceived serviceability issues.

1.2: Scope and Objectives

The investigation is focused on a typical girder in the south upper deck from which a large number of the letters of concern were reported. This girder is selected because it supports a portion of the student seating section; a demographic likely to participate in the rhythmic jumping. A global investigation of the south upper deck is not warranted because there is little continuity from one section to the next making it unlikely for vibrations to carry-over into adjacent sections. Therefore, information gathered along this girder should shed light into the behavior of the rest of the structure. Prior to the 2007 football season, an array of accelerometers and tilt-meters were installed at key positions along the girder. The position of these sensors was unchanged throughout the season. The in-service vibrations were recorded for all home games of the 2007 football season.
Although some letters of concern did originate from spectators seated in the north upper deck, this area is not investigated in detail. The north upper deck is constructed using cast-in-place girders that are substantially stiffer than the post-tensioned girders from which the south upper deck is constructed. The additional mass and stiffness of the cast-in-place system makes this area less susceptible to vibration issues and is therefore not considered to be a high-risk area. As a short term solution during the data collection period of the research, both the south and north upper decks are monitored in real-time from the sound-booth. If accelerations above 40 milli-g’s are experienced, the music causing the in-time jumping can be shut off and the vibrations are quickly damped-out.

The dynamic testing of the stadium is conducted using output-only system identification techniques due to limits in budget and lack of an adequately strong and measureable excitation source. Static testing of the stadium is not conducted due to the lack of a stationary reference point from which displacements can be measured. The major objective of dynamic testing is to obtain accurate and reliable frequencies, mode shapes, and damping ratios for use in model updating.

The finite element model is created as a 2-dimensional model to eliminate complication in the model updating portion of the research. Due to the simple connections between the girders and the z-slab decks, out of plane motion is anticipated to be minimal. The lack of out-of-plane motion warrants the restriction of the model to a 2-dimensional structure.
Model updating aims to provide a reasonable correlation between the model’s modal characteristics and the experimentally determined characteristics. For the updating to be considered a success, the updated model must be physically meaningful, with parameters that can be rationally explained. The updated model will then be further validated by comparing analytically obtained results to results obtained from stadium monitoring. Allowances for differences between assumed loading and actual loading should be made when assessing the degree of correlation between the two responses.

The end result is a well validated model that can be used to determine how the stadium will respond to loadings that have not yet been experienced; examples of which include: rhythmic loads at various tempos, higher levels of crowd participation, and new crowd configurations.

1.3: Outline of Thesis

The research that follows is separated into five additional chapters. The first section (Chapter 2) is a thorough review of literature related to the topic. The literature review provides insight into the origin, development, and state-of-the-art for each of the methodologies by which this research is conducted. Serviceability issues related to human induced vibrations are discussed as they relate to various structure types and specifically stadia. Previously installed vibration monitoring systems are discussed with a focus on the needs of system installed in Clemson. The system identification tools used to extract the dynamic characteristics of structures are examined, paying close attention
to output-only techniques used in this research. Finally, model updating techniques are examined with a special focus given to sensitivity based, parameter updating techniques.

The focus of Chapter 3 is on the field study of Clemson’s Memorial Stadium. Information on the geometry, construction methods, and photographs of the south upper deck are detailed therein. Additionally, the selection of a monitoring system and details of the installation are included in this chapter. Finally, comments on the in-situ data collection conducted throughout the research are made.

The information collected in Chapter 3 is then used extensively in Chapter 4. Chapter 4 begins with the initial FEM creation. This section outlines all of the thought-processes, parameter value selection, and modeling assumptions that go into the creation of the initial finite element model. The next section undertakes the task of identifying the dynamic properties of the stadium. The Frequency Domain Decomposition (FDD) Method of identifying modal parameters is validated using an analytical model. Upon successful validation, the FDD method is used to extract the stadium’s modal characteristics from the previously recorded acceleration time histories. Finally, the modal characteristics are used to update structural parameters of the initial FEM so that the modal characteristics of the final model match the experimentally determined values, thus producing a more reliable model.

The final research oriented chapter, Chapter 5, uses the updated model to determine the response of the girder to various loading scenarios. The loading scenarios chosen are
based on loadings experienced during stadium monitoring. By doing this, the analytically
obtained response using the updated model may be compared to the experimentally
obtained response in hopes of validating the model. Once validated, the model can be
used to investigate loading scenarios that have not yet occurred.

The entirety of the research is summarized in Chapter 6. Observations are made
regarding each of the tools used in the research. The success and limitations of those
tools is also described. Additionally, recommendations related to Clemson’s Memorial
Stadium are made and finally, a few suggestions for future research are expressed.

The reader may also find the appendices helpful in aiding in their own research or in
better understanding the exact methods used in conducting this research. Included in the
appendices are the MATLAB scripts used to implement the previously mentioned tools
and the text input files from which the initial and updated finite element models are
created.
2.1: Occurrences of Human Induced Vibrations

As structural systems become increasingly efficient, the likelihood of serviceability issues due to human induced vibrations has become more widespread. Human induced vibrations, as the name indicates, are motions in a structure or component of a structure caused by interactions between people and the structure. The two main sources of human induced vibrations are pulse loads and rhythmic loading. A pulse load is a load that is applied nearly instantaneously. An example of a pulse load is a person jumping off of a footstool. The pulse load will cause vibrations to occur as the system tries to return to static equilibrium. Rhythmic-based human-induced vibrations are the more common of the two main sources. Typical causes of rhythmic loading are: walking, dancing, exercise classes, or excited crowds. Rhythmic loading by a crowd is generally coordinated by a musical cue. When a cue is used to set the tempo, the loading has two components: a synchronized component and a stochastic component (Parkhouse and Ewins 2006).

Serviceability issues from human-induced vibrations have been documented in numerous types of structures including: footbridges, residential structures, multiuse commercial, and stadia. One of the most notable recent instances of a bridge structure is the Millennium Bridge in London (Pavic 2002). Lateral movement caused by pedestrians walking on the bridge forced the bridge to temporarily close until a retrofit could be
installed. Numerous other bridges have had similar issues adding to the rich knowledge base on the subject (Dallard et al 2001). Due to the fact that bridge structures are designed with large loads, the stiffness of a bridge is large enough that a human-induced pulse load typically will not have a large enough magnitude to cause a noticeable response in the structure.

Along with bridges, poorly designed wooden residential floor structures are notorious for their susceptibility to vibrations. Unlike bridges, wooden floors are susceptible to pulse type loads in addition to rhythmic or harmonic loads. The combination of architectural demand for longer spans and more efficient cross sections (I-joists) have, as a consequence, provided more flexible systems. These issues have become so widespread that numerous articles on the prevention of floor vibrations have been published providing rules of thumb and design tools to prevent such phenomena (Dolan et al 1999 and Woeste 2000).

In much the same way as wooden floor systems, mixed-use properties are susceptible to both rhythmic loading and pulse loading. Any rhythmic loading occurring in one area of the building can propagate elsewhere. If the structural system is not stiff enough, or does not have enough damping, the vibrations can become bothersome to others in the vicinity. In one instance, recalled by Dr. Thomas M. Murray of Virginia Tech, rhythmic loading from dance studios in a 10-story tower caused complaints from the office staff (Murray 2008). The rhythmic loading from studios on the fourth, seventh, and ninth
Floors could all be felt to varying degrees by the office staff on the tenth floor. Modern office buildings have moved to lighter weight cubicles rather than built in partitions reducing the damping due to nonstructural elements. Without a high damping ratio, vibrations are free to propagate around the building. Case studies of walking induced vibrations involving offices, classrooms, and clothing stores have also been documented (Hanagan 2005).

When large, dense crowds are present; vibration levels can go beyond serviceability issues and become structural issues. Stadia and concert venues are two locations where crowds are extremely dense, excited, and coordinated enough to cause high magnitude vibrations. In addition to a strong load, the design of modern stadia make them more susceptible to vibration issues. Modern stadia are being constructed with lighter and more slender structural members and demand for completely obstruction free sight lines have eliminated the ability to use columns in some locations (Reynolds 2004).

Vibration issues have been reported at Camp Randall Stadium at the University of Wisconsin from the playing of the song, *Jump Around* (Tuan 2004). The stadium is a concrete frame with a large portion of the main girders cantilevered. The jumping of the crowd caused enough concern that administration monitored the situation and eventually decided to discontinue the playing of the stimulating song. More recently, Bright House Stadium at the University of Central Florida has experienced vibration issues from the in-time bouncing of students to the song, *Kernkraft 400*, earning the new stadium the
nickname, “The Trampoline” (Zaragoza 2007). Concerns have caused administration to monitor the stadium and limit the number of times per game the song is played.

Excessive motion has been reported at Elland Road Stadium in Leeds, United Kingdom (Reynolds 2006) and Cardiff Millennium in London (Rogers 2000) both of which have large cantilever spans and natural frequencies that can be easily activated by human induced vibrations. In each case, crowds moving to a musical or visual cue caused high levels of vibration to occur in the stadium causing concern from patrons and administration officials.

Similar to Bright House Stadium, Frank Howard Memorial Stadium at Clemson University has received letters of concern regarding movement caused by human induced vibrations. The in-time bouncing of spectators in the upper deck due to the playing of Zombie Nation’s song, *Kernkraft 400* has caused enough motion to cause concern from other spectators not participating in the bouncing. The majority of the letters of concern have come from spectators located in the south upper deck which is a precast concrete structure with portions of the structure cantilevered. A portion of the south upper deck provides seating for students who are typically more responsive to musical stimulus increasing the possibility of serviceability issues.
2.2: Vibration Monitoring

Advances in the collection of acceleration time histories of major civil structures have been driven by the field of structural health monitoring (SHM). Structural health monitoring utilizes system identification techniques to extract dynamic properties such as natural frequencies, mode shapes, and damping estimates in an effort to monitor changes in structural behavior over its life. The quality of the dynamic properties extracted is directly linked to the quality of the data collected. For applications involving human induced vibrations, a monitoring system should be able to collect the following information:

- Vibration information for both occupied and empty conditions.
- Crowd information including size, distribution, and activity level.
- Environmental data including wind speed/direction and air temperature.

Vibration data collection can be used for three purposes: real-time monitoring of an occupied stadium, for system identification purposes, or to investigate human-structure interaction. An acquisition system can be designed specifically for each application or versatile enough to be used for all three applications. A system for real-time monitoring may only need accelerometers at locations of anticipated large response in order to alert officials of an exceedance rather than a complete array at many locations. For such a system, the only necessary equipment is an array of accelerometers, a signal processing
unit, and a user interface for monitoring purposes. The data is processed and monitored in real-time and therefore does not need to be recorded nor is remote access needed. Before the start of each occupancy, the system should be enabled, and deactivated after the crowd has dispersed.

When data is to be used for system identification, the equipment needed is dictated by the type of system identification being utilized. For an empty stadium, there are two main types of system identification: input-output system identification and output-only system identification (SAMCO 2006). Input-output, commonly called forced-vibration, uses an excitation source to produce a known forcing function. Engineers are able to measure the response of the structure at key points to create a series of frequency response functions from which modal properties can be extracted. For smaller structures, an impact hammer can be used to produce the excitation source (Merkley 2007). Impact hammers are limited by the amount of energy they can impart to the system, limiting the number of modes that can be excited.

For larger structures, electrodynamic shakers or eccentric mass vibrators may be used. As opposed to an impact hammer, these allow for more control of the amplitude, type, and frequency content of the imparted load. The difficulty with these machines is that they are expensive and cumbersome to maneuver. Additionally, for extremely large structures, standard shakers are unable to produce forces large enough to adequately excite the structure. Long span bridges, high-rise buildings, dams, and large stadia are all
examples of such structures. When a shaker is not able to excite the stadium, output-only methodologies can be used.

Output-only system identification techniques do not use an artificial means of excitation. Instead, naturally occurring vibrations caused by wind, traffic, or occupants are recorded. Instruments to record the response of large, flexible structures, such as long span bridges and tall buildings, have been available for some time. The typical response of these types of structures to ambient excitation is large, noticeable, and easily measured. It has only been in recent years that modern force balance accelerometers have made it possible to record low magnitude, low frequency accelerations (SAMCO 2006).

Typical system identification applications involve the placement of a few reference sensors on the structure to be used in combination with a number of moveable sensors that can be relocated as needed to improve spatial resolution. For long term monitoring, redeployment is not desirable making the initial location choices critical and may require more sensors than a moveable system. In either case, the analog signal created by the instruments can be transmitted along cables to a standard PC. Analog to digital conversion is required with a minimum desired resolution of 16 bits. For projects where it is not practical to run a cable between instrument and data collection point, wireless transmission of data or local digitization have been implemented with success (Cunha 2001 and Rodrigues 2004).
2.3: System Identification Techniques

Originally developed in the aerospace and systems control industries, system identification is now commonly used in mechanical, electrical, and structural engineering. The aim of system identification, as it relates to civil engineering, is to accurately identify the dynamic characteristics of a built structure. The dynamic characteristics needed to be identified are typically the natural frequencies along with the corresponding mode shapes and damping ratios. As previously mentioned, the system identification method chosen will be dictated by the method used to gather the response time histories of the structure.

For response time histories gathered using a known input (forced vibration), the system identification technique will fall into a class of techniques known as “input-output modal testing” (SAMCO 2006). The general principle behind these methods is to perform a curve fitting between the measured and the theoretical functions utilizing frequency response functions or impulse response functions. Since this paper does not deal directly with input-output modal analysis, further detail on the subject is not provided. A complete resource on the subject of forced vibration testing can be found in D.J. Ewins’ text, *Modal Testing- Theory, Practice and Application*. The remainder of this section will focus on output-only system identification techniques.

Output-only techniques are categorized by the domain in which they operate; be it, time or frequency. Algorithms set in the time domain work by choosing a mathematical
model (typically time discrete state space stochastic models, ARMAV or ARV models) and vary the modal parameters until the analytical response matches the experimental response. One method frequently used to evaluate these functions is the Vector Random Decrement Method (Asmussen et al 1999). This method, an improvement on the Random Decrement Method, takes the ambient response of the structure and transforms it into Vector Random Decrement functions from which modal parameters can be extracted using free decay analysis. Other techniques used for output-only system identification are extensions of input-output methods with the assumed loading to be zero mean Gaussian white noise (SAMCO 2006). A few examples include the Polyreference Complex Exponential, the Least-Squares Complex Exponential or the Ibrahim Time Domain Method. Furthermore, some time domain methods have been adapted and applied in the frequency domain with success such as the Eigensystem Realization Algorithm (ERA) (Juang and Suzuki 1988). The ERA method is a well tested and generally accepted method of system identification.

Frequency domain techniques have become increasingly common because of their ease of application and ability to visualize the results. The most fundamental of these methods is the Peak-Picking (PP) method (Peeters and Ventura 2003). This method creates an output spectrum matrix whose peaks indicate the location of natural frequencies. This method assumes low damping and well spaced modes. If modes are nearly coincident, the two modes will appear as one peak. If two modes are able to be identified, the results
will be contaminated by the neighboring mode (Brinker et al 2001). An enhancement on PP can be found in the Frequency Domain Decomposition Method (FDD).

The FDD method improves on PP by performing a singular value decomposition on the output spectrum matrix (often called the spectral matrix) (Brinker et al 2001). Three advancements of the FDD are (Gentile and Gallino 2007):

1. The SVD’s ability to separate signal space from noise space and easily produce mode shapes.
2. The ability to detect closely spaced modes without a loss of clarity.
3. Damping estimates can be obtained through an enhancement to the method often know as the Enhanced FDD (EFDD).

The EFDD method produces damping ratios by isolating the autospectral densities near a resonant peak, zero pads it, and applies an IFFT to create an autocorrelation function that takes the form of free decay in the time domain. Damping ratios can be identified from the free decay using simple logarithmic decrement techniques. The ability to produce damping ratios is a tremendous improvement on PP. The FDD method has become a standard in output-only techniques having been successfully used in a number of studies and implemented in commercially available software (Magalhaes et al 2007, Gentile and Gallino 2007, Brinker et al 2001, Michel et al 2008, ARTeMIS 2008).

The benefits and weaknesses of all of the system identification techniques in existence is a large enough topic to dedicate an entire volume. As a result, an interested reader is
directed to the numerous works that compare and contrast the available techniques
(Peeters and Ventura 2003, Magalhaes et al 2007, Bernal and Beck 2004, SAMCO 2006,
He and Fu 2001)

2.4: Finite Element Model Optimization

The current nature of design and analysis necessitates the need for assumptions to be
made about the nature of a structure and its components. Boundary conditions,
connection fixity, material properties, construction tolerances, and manufacturing quality
can all play a role in the final behavior of a structural system. In design, simplifying
assumptions are often made; airing on the side of caution. Analysis, on the other hand,
requires an amount of accuracy that is not typically available in an initial finite element
model (FEM). Once erected, a structure can be tested by static and dynamic means,
revealing the true behavior of the structure. Without fail, the predicted and experimental
response of the structure will vary to a degree. Past studies have found ranges of
differences between analytical and experimental values of: 8-44% (Ventura et. al 2005),
1-16% (Zhang 2000), 0-17.4% (Zhang 2001), and 4-20% (Jaishi and Ren 2005). It is the
desire of the engineer to update the FEM in such a way, that it more accurately portrays
the behavior of the actual structure. There exist infinite combinations of changes that can
be made to the FEM in order to obtain a better correlation; making it a task that cannot be
accomplished by human insight alone. The field of structural optimization provides a
rigorous manner by which parameters are selected and manipulated such that the resulting FEM is both more accurate and physically meaningful.

Initially developed in the mechanical and aerospace industries, the first survey of structural optimization as it applies to structural dynamics was completed by Mottershead and Friswell in 1993 and is recommended reading for introduction into the field. The methods of model updating can be split into two main classifications: direct optimization and iterative optimization (Jashi and Ren 2005). In direct optimization, the system matrices (mass and stiffness matrix) are altered directly such that the resulting eigenvalues more closely match the experimental results. The direct optimization problem may be solved using Lagrange multiplier techniques such as: The Reference Basis Method, Matrix Mixing Technique, Eigenstructure Assignment, or Inverse Eigenvalue Methods (Mottershead and Friswell 1993). Each of the techniques have strengths and weakness with all procedures working well for simple structures. These methods become flawed when applied to more complex systems.

In previous work, simplifying assumptions, such as a mass matrix assumed invariable and known, have been made; which is not a valid assumption in practice (Caicedo et al 2004). Direct optimization cannot handle the case where mass and stiffness matrixes are coupled; as is the case with suspension bridges (Zhang 2000). Also, the resulting mass and stiffness matrices typically become physically meaningless (Berman 1971) and great effort must be taken to maintain structural connectivity (Jaishi and Ren 2005). A
physically meaningless model may exhibit the correct response for the criteria it was
validated against but may not perform correctly under new loading functions making it
useless for future applications. Berman (1971) concluded that is impossible to identify a
physically meaningful model by using a direct approach (Mottershead & Friswell 1993).

The more commonly accepted method of model updating involves an iterative process.
Sometimes called a “local method”, physical parameters within the FEM are altered
rather than the system matrices. Instead of changing the stiffness term in a matrix,
Young’s modulus or the moment of inertia of a member is altered in order to alter the
stiffness matrix. As a result of updating parameters, the symmetry, positive definiteness,
and sparseness of the stiffness matrix is implicitly maintained. Additionally, the physical
meaning of the resulting changes can be immediately seen after updating is complete,
providing the analyst a justification for the nature of the new model. The choice between
direct methods and iterative method is a balance between the effort required and the
accuracy obtained.

A number of iterative updating methods exist with most using the sensitivity of the
parameters to update the model. The sensitivity approaches use a common set of
equations with the main differences being categorized by (Doebling et al 1996):

1. The objection function that will be minimized
2. The parameter constraints
3. The numerical algorithm used to execute the optimization
The objective function aims to give weight to the deviation between the analytical model and the experimental values. In simplest of terms, this function should be zero if the model matches perfectly to the experimental values and increase as the deviation increases. The objective function can be any combination of the frequency residual, a mode shape related function, or modal flexibility residual (Jaishi and Ren 2005). Early implementation of model optimization commonly used an objective function based solely on the frequency residual (Brownjohn and Xia 2000, Zhang 2000, 2001).

Seeking a more robust objective function, current research has included the use of other residuals in hopes to expand model optimization techniques for use in structural health monitoring and damage detection techniques. It has been shown that for large civil structures, frequency residual alone is adequate in model optimization but frequencies can be insensitive to local damage and thus, mode shape related functions are used in structural health monitoring techniques (Hemez 1993). For an objective function composed of a combination of residuals, weighting factors are commonly used to either give equal-weight or variable-weight to each residual.

The analyst can use any means to calculate residuals that they so choose, but a common set of equations has become typical in this field (Jaishi and Ren 2005):

$$\Pi_1 = \sum_{i=1}^{m} \alpha_i \left( \frac{\lambda_{ji} - \lambda_{ei}}{\lambda_{ei}} \right)^2$$  

Frequency Residual  

(2.1)
\[ \Pi_2 = \sum_{i=1}^{m} \beta_i \left( 1 - \frac{\sqrt{MAC_i}}{MAC_i} \right)^2 \]  
Mode Shape Residual \hspace{1cm} (2.2)

\[ \Pi_3 = \frac{m}{n} \cdot \sum_{j=1}^{n} \left[ \frac{u_{aj} - u_{ej}}{u_{ej}} \right]^2 \]  
Modal Flexibility Residual \hspace{1cm} (2.3)

For the frequency and mode shape residuals, \( \alpha \) and \( \beta \) are the weighting coefficient mentioned earlier. \( \lambda_{ai} \) and \( \lambda_{ei} \) are the analytical and experimental natural frequencies for the \( i^{th} \) mode, respectively. The Modal Assurance Criterion (MAC), originally proposed by Allemang and Brown in 1982, is a commonly used means to compare the \( j^{th} \) analytically obtained mode shape, \( \Phi_{aj} \), with the \( k^{th} \) experimentally obtained mode shape, \( \Phi_{ek} \), as defined:

\[
MAC_{jk} = \frac{\left| \Phi_{ek}^T \Phi_{aj} \right|^2}{\left( \Phi_{aj}^T \Phi_{aj} \right) \left( \Phi_{ek}^T \Phi_{ek} \right)}
\]  
(2.4)

The MAC ranges between 0.0 and 1.0, with a value of 1.0 being a perfect match. A perfectly correlated model would have a MAC matrix in the form of an identity matrix. It should be anticipated that for closely correlated models, when \( k=j \), a MAC value near 1 should be obtained. The MAC is independent of the manner in which the mode shapes are normalized and can handle complex modes if the transpose is taken as the conjugate transpose (Friswell and Mottershead 1995). Additionally, the MAC should be used when only a frequency residual is considered to make certain that the frequencies being compared are for the same type of mode (bending, torsion, or otherwise) (Friswell and Mottershead 1995). This is especially important for closely spaced modes.
description of the modal flexibility method, which is used in the field of damage
detection and structural health monitoring, can be found in Zhoa and Dewolf, (2002).

Once the objective function has been defined, the next step in model updating using the
iterative method is the selection of the parameters that will be used in the updating. A
complete FEM will have hundreds of individual parameters that can be updated. It is
important to keep the number of updating parameters to a minimum or else the problem
may become ill-conditioned (Friswell and Mottershead 1995). A number of factors go
into selecting appropriate parameters as laid-out in Brownjohn and Xia, (2000).

The parameters selected should have uncertainty in their values. A parameter that is
fairly well known (the Young’s Modulus of steel, for example) should not be included as
an updating parameter because the change in that value may lead to the correct model
behavior for the wrong reason, invalidating the model. The sensitivity of every
parameter that is believed to possibly impact the dynamic behavior of the model should
be tested. A sensitivity analysis is a simple test whereby parameters are altered one at a
time and the impact on the frequencies is measured. Parameters that have the most
impact on the model frequencies should be considered in the updating process. The
process of selecting parameters is not without engineering judgment; if frequencies are
not terribly sensitive to a parameter believed to play a role in the behavior, that parameter
should be included in the updating regardless. Finally, parameters of components not
directly measured during dynamic monitoring, that impact the behavior of the monitored
portion of the structure can be included. For a cable-stayed bridge, the deck may be the only portion monitored, but the stay-tower has a direct impact on the behavior of the deck and its properties should be considered in the updating process. A parameter weighting matrix may be introduced into the optimization problem if the analyst wishes to specify the amount of uncertainty they believe is in each parameter value. By doing this, they can guarantee that parameters that are known with relative certainty change less than those that are more uncertain (Friswell and Mottershead 1995).

Optimization schemes are purely mathematically driven, and as a consequence, the parameters selected to satisfy the problem may become physically impossible. While negative values or extreme variations in the parameter may minimize the objective function mathematically, they will not provide a meaningful model for future use. To prevent the algorithm from converging on a physically meaningless model, parameters are often constrained to realistic values. Constraints can be added in two ways: either implicitly or explicitly (Doebling and Farrar, 1999). Implicit constraints are the relationship between the parameters and how they impact the system matrices and are handled internally by a structural analysis program. Explicit constraints are applied to restrain the possible perturbations of the parameters to within certain limits. The limits of the perturbations are often dictated by the confidence in the initial parameter value. For direct updating, constraints can be placed on the system matrices such that the matrix remains symmetric, sparse and positive definite. The use of constraints is a trade-off
between the accuracy of the model and having a physically meaningful model for future use.

Due to the objective function being a non-linear function of the parameters, and a lack of a practical means to solve this non-linear function, the solution to the problem requires a linear approximation be made and the problem solved iteratively (Friswell and Mottershead 1995). The most common manner to linearize the problem is to represent the objective function (or penalty function) as a truncated Taylor series expansion relating the modal data as a function of the parameters. By truncating the Taylor series to only the first order effects, a simple approximate relationship is formed. The cost of this approximation is that the FEM must be reevaluated after each iteration; making this a time-consuming process. A more complex relationship must be made when the number of parameters considered is much larger than the number of measurements made. To solve a rank-deficient problem, a weighted least squares technique is appropriate (Friswell and Mottershead 1995). In addition to the least squares technique, the problem may be solved by the Minimum Norm Method which employs singular value decomposition or can be solved with the Moore-Penrose Inverse (Friswell and Mottershead 1995). To speed up convergence, the Conjugate Gradient Method may be used which eliminates finite difference numerical estimation (Giraldo 2006).

When more parameters than measurements exist, the problem can be solved by an infinite combination of parameters. The best solution to the problem is typically considered the
solution that produces the smallest change in the parameters (Friswell and Mottershead 1995). The assumption of the smallest change in parameters being the most correct model does not allow for engineering judgment into what is the best combination of parameters. To provide the analyst with more freedom in the selection of the best results, a variety of initial parameters, response values, and constraints should be evaluated (Brownjohn and Xia 2000). The combination of parameters that is most justifiable can then be used to create the updated model. The process of evaluating multiple combinations of parameters can become time consuming. The “hop, skip, and jump” method can be used to automate the exploration of multiple alternatives (Zárate and Caicedo 2008) resulting in a more efficient process.

The process of optimization is complete when the objective function has reached a threshold or an iteration limit has been reached. Divergence can occur if the initial residual is too large; in which case, the initial model (sometimes called the ID-Model) should be reconsidered and altered as necessary (Brownjohn and Xia 2000).
CHAPTER THREE

FIELD STUDY OF CLEMSON MEMORIAL STADIUM

3.1: Stadium Geometry and Construction Methods

Clemson Memorial Stadium, like most stadiums in collegiate sports, has grown along with the university. The most notable additions to the stadium came in 1978 and 1983 with the addition of a south upper deck and a north upper deck, respectively. Figure 3.1 provides a plan view of the stadium as it is in 2008. The south upper deck is constructed using a precast, post-tensioned concrete structural system. This system was selected after construction delays eliminated the option of using a traditional cast-in-place system (Coleman 1981). The north upper deck, in contrast, was constructed with a traditional cast-in-place concrete structural system. By using a post-tensioned system in the south upper deck, engineers were able to provide a more efficient system in terms of structural weight as compared with the north upper deck. A visual comparison between the main girders of the south and north upper decks shows a drastic increase in member size on the north side as shown in Figure 3.2. The north upper deck’s increase in mass and stiffness due to joint fixity and deeper members decreases its susceptibility to vibration issues. Consequently, it is of little surprise that the majority of the letters of concern, caused by the playing of Zombie Nation’s song, Kernkraft 400, came from spectators seated in south upper deck. For this reason, this research focuses on the human induced vibration susceptibility of the south upper deck.
The south upper deck is broken up into 18 typical sections. Each section is composed of precast z-slab decking resting on precast, post-tensioned, non-prismatic, concrete girders.

![Plan View of Memorial Stadium in 2008](image)

Courtesy of Clemson Athletic Ticket Office

Figure 3.1: Plan View of Memorial Stadium in 2008.

The double girders are composed of three pieces that were erected in place and post-tensioned together to form a continuous member. A seven-inch air space separates the parallel girders which are supported at three locations atop cast-in-place columns as shown in Figure 3.3. The location of the three columns makes it such that the girder is
Figure 3.2: Typical girder types (a) South Upper Deck  (b) North Upper Deck
cantilevered out at both ends. By cantilevering the girder at the bottom end, architects ensured that columns would not interfere with the sight lines of the spectators below. The z-slabs on either side of the column line are supported by one of the girders making up the double girders, making the sections of the stands more isolated from one another as can be seen in Figure 3.4. The parallel girders have continuity at seven locations along their length provided by mechanical and concrete closure pours. The girder to girder connections, depicted by the gray areas in Figure 3.3, occur at the tops of columns, the ends of the girders, and at the two construction joints where the pieces are post-tensioned together. In addition to the slabs and girders, precast railings exist along the top and bottom of the stands adding significant point loads at the cantilevered tips of the girders.
Figures 3.5 and 3.7 show the bottom and top precast handrails, respectively. Finally, portals giving access from the concourse to the seats occur in every other section of the stands. As seen in Figure 3.6, the wall on one side of the portal is both built on top of and hung from the girders, adding significant mass. However, due to the cold joints between the pieces these wall sections add little stiffness to the girder.

Figure 3.4: Cross-Sectional View of Double Girder.
Figure 3.5: Bottom Precast Handrail

Figure 3.6: Girder at Portal

Figure 3.7: Top Precast Handrail
3.2: Instrumentation Installation

The configuration of the instrumentation is highly dependent on engineering judgment. The objective of the instrumentation is to mount instruments in locations and orientations that provide the most complete understanding of the dynamic behavior of the system as possible. To do this, a list of all possible forms of motion was created. This list includes:

1. Bending of the girder about its major axis
2. Horizontal sway (parallel to the girder) of the frame substructure supporting the girder
3. Differential horizontal motion between the girder and top of a column
4. Joint opening occurring at the construction joints
5. Torsion of the girder
6. Differential movement between the double girders

Using the identified types of movement, a strategy to capture the motion of each is determined.

The first type of movement to be captured is the major axis bending of the girders. The objective of capturing this motion is to capture the frequencies and mode shapes associated with the major axis bending. To do this, accelerometers must be installed in the optimal location to capture the anticipated motion. Linear motion sensors (LVDT) could also be used in determining the frequencies and mode shapes but requires a
stationary point of reference. The anticipated deflections to be measured are small compared to the nearest stationary reference point, limiting the precision that can be obtained. Measuring a fraction of an inch from a hundred feet away would not provide reasonable data, thus, the use of accelerometers to obtain the desired information.

![Figure 3.8: Instrument Layout](image)

With a limited number of accelerometers and the inability to redeploy the instruments, it is crucial that the location of the instruments maximize the information that can be obtained. In setting up the array, the location of each sensor is chosen so that it will correspond to areas of maximum displacement during all of the modes. Ensuring that the sensors are in locations of large displacements provides a response time history with a strong signal to noise ratio, making for easier data processing.
Additionally, accessibility is a major concern in determining where to locate sensors. The upper tip of the girder is approximately 60 feet above the concourse and 115 feet above grade making access available only by rappelling off the top of the stadium. The use of rope only allows access to the very tip of the girder. To gain access to other locations along the length of the girder, an articulating boom lift was used. Due to height limitations of the boom lift, instruments could not be placed higher than column line C with the exception of the tip of the girder. To understand the bending of the girder about its major axis, accelerometers 0, 1, 4, 7, 8 and 9 were installed. The locations of these sensors met both the accessibility criteria and are locations of perceived maximum response. The sensors deployed in the horizontal direction are expected to have small magnitudes compared to their vertical counterparts and are thus sensitive to low signal to noise ratios but will give an idea of the movement in the horizontal axis.

The instruments used in data collection play a crucial role in the collection of accurate and meaningful data. The first step in any data collection process is the instrument itself. Crossbow accelerometers, CXL02LF1Z (single axis) and CXL02LF3 (tri-axial) were used to measure accelerations and Crossbow tiltmeters, CXTLA01, were used to measure rotation. They accelerometers have a range of ± two g’s with a voltage output range of zero to five volts with an output of 2.5 volts under no acceleration. The sensitivity of the instruments is one g per volt. For accelerometers oriented vertically, a constant one g is registered. To apply a gain to the system, the constant one g was offset so that the gain multiplication did not overload the voltage range. To do this, the signal passed through a
signal conditioning board prior to analog/digital conversion. In addition to zeroing the signal, the signal conditioning board also applied an electronic gain of 20. Being that the anticipated maximum accelerations were on the order of 0.1 g’s, a gain of 20 would increase a reading of 100 milli-g’s to a voltage of 2 volts. Having a stronger signal increased the resolution of the system during the A/D conversion. Finally, the signal conditioning board applied a low pass filter to the system. The anticipated frequency range of the stadium is below 15 Hz. The frequency content associated with electronic noise is of much higher frequency content. The use of the low pass filter eliminated the high frequency harmonic portion of the recorded signal.

After passing through the signal conditioning board, the analog to digital (A/D) conversion of the signal took place. The game data was processed with a 16-bit, National Instruments crRIO-9215 analog to digital converter module housed in a NI cDAQ-9172 chassis. As mentioned earlier, the use of the gain aided with the resolution of the A/D conversion. After the implementation of the gain, a resolution of 0.016 mg is achieved. This means that the true measurement is only within 0.008 mg on either side of the value recorded. This resolution is more than adequate for recordings made during the game, but for ambient vibration testing, a more refined A/D conversion is needed.

To accurately create a model of the stadium, zero occupancy acceleration data is needed. In traditional testing, the crowd is used as the excitation source. In addition to adding a source of load, they also increase the system mass in a non-uniform manner which is
difficult to identify and model. To create a more accurate model, ambient vibrations (vibrations caused by wind and other natural loadings) are used. Due to the extremely low level of excitation associated with ambient vibration testing, the magnitude of the accelerations being recorded are on the order of a 3 to 5 milli-g’s. For a 5 milli-g recording at a 16 bit resolution, the number of “buckets” available is 312. Accelerations of the magnitude of 40 milli-g’s provides 2500 buckets; a much more substantial relative resolution.

To increase the resolution of the system for the case of ambient vibration testing, a Data Translation DT9837 A/D converter was used. This system can handle a maximum of four channels of data. For this reason, only the four vertical accelerometers were used for ambient vibration testing. The resolution of the 24 bit A/D convertor is 0.001 milli-g’s. For low excitation levels, the additional clarity of the signal greatly increases the likelihood of capturing its harmonic content.

To transform the voltage signal into engineering units, National Instruments LabVIEW 8 data acquisition software was used. The signal was divided by the gain before the sensitivity (conversion from volts to g’s) was applied. Once converted to engineering units, the signal was saved into ten minute segments in units of g’s. The sampling frequency used is important when dealing system identification. In order to prevent aliasing of the signal, the sampling frequency must be at least two times the anticipated signal frequency (Doebling and Farrar 1999). This concept is known as the Nyquist-
Shannon sampling theorem and is a typical starting point in determining a sampling frequency. In order to guarantee that a data rich signal is captured, a sampling frequency of 200 Hz is selected. For use in real-time monitoring of the signal, a plot of each of the data channels was displayed in a window in units of milli-g’s. This step allowed engineers to monitor any anomalies in the signal and to also add notes about each data file.

To measure the horizontal rocking of the frame, accelerometers 2 and 6 are installed at the top of columns B and C. Column A is an extremely short column and will have very little horizontal motion providing little useful data. The horizontal accelerations at the tops of the columns will be used to establish appropriate boundary conditions for the girder model. If large horizontal accelerations are experienced at the top of the columns, then the motion of the girder is dependent on the substructure. In this case, the substructure will need to be modeled in order to accurately depict the stadium’s response to future loadings. For the case of low horizontal accelerations, the top of the columns can be considered to be a fixed condition and the substructure need not be modeled. In addition to the rocking of the substructure, accelerometers 2 and 6, in tandem with accelerometers 3 and 5, provide an indication of differential sliding between the column and the bottom of the girder. The girder is bearing on a ¼” neoprene pad but is also substantially connected to the top of the column providing little insight into the anticipated behavior at that location.
Precast construction typically does not provide the amount of connection fixity as cast-in-place construction. One such condition that is to be investigated in this structure occurs at the construction joints between the three girder pieces. To check if the construction joint is “opening” during the vibrations, a set of tilt-meters (instruments 10 and 11) are installed on either side of the construction joint. By comparing the relative rotation on either side of the joint, the continuity at that location can be assessed. For the data acquisition of the tilt-meters, the data is processed in the same manner as the acceleration data with the exception of the sensitivity being changed to 0.101 degrees per volt.

Torsion along the axis of the girder has little impact on the movement perceived by the occupants of the stadium and is therefore of little concern in that aspect. The main purpose of capturing the torsional motion of the girder is in the processing of natural frequencies. Examining the frequency content of the vertical accelerometers, one is unable to distinguish a natural frequency associated with a torsional mode from a pure bending mode. In the initial investigation, it is believed that the torsional modes associated with the girder are outside of the frequency range under investigation and thus the torsional motion is not to be directly sought.

The final type of motion to be analyzed is the differential movement between the two girders. The amount of differential movement is a direct function of the degree of fixity between the two girders. A large amount of fixity means that the girders act as if they were one member, whereas, little fixity would allow the members to move independently.
of one another. In the initial investigation, the amount of fixity present between the two members appears to be high with numerous continuity points, indicated by grey areas in Figure 3.3, along its length. Due to the high amount of fixity, it is believed that the girders will share load, acting as a single girder. Therefore, differential movement between the two girders was not measured.

The instrument array is installed at the beginning of the 2007 football season using a combination of rappelling and an articulating boom lift. The instruments were mounted on brackets which are mechanically attached to the girder. Once the instruments were tested to ensure functionality, any instrument at risk of damage due to the elements was covered with a weatherproof case and caulked shut. The wiring was run between the two girders down to a central data collection center set up in a storage space adjacent to the girder being monitored. The wires were run through the girder as inconspicuously as possible as to not draw attention to the sensors and protect them from abuse.

3.3: In-situ data collection

Throughout the 2007 football season, data is collected at each of the home football games. The data collection system is initialized 30 to 60 minutes prior to the kickoff. Initializing the system well before the stadium becomes completely occupied provides a look into how the addition of the crowd impacts the way in which the structure behaves. The time histories recorded by the instruments are saved into ten-minute long individual
text files. Included in the text files are the time histories in g’s or degrees, the start time of the file, the date, and the sampling frequency. In addition to the automatically obtained data, weather, crowd, and game information are recorded for future reference.

Variations in structural behavior are to be expected due to changes in temperature and this information will help in explaining small variations in behavior. The crowd size and dispersion are also crucial to understanding the motion of the stadium. The crowd changes the systems behavior in numerous ways including the addition of significant mass, altering the damping ratio, and providing various loading scenarios (rhythmic, reactionary, and ambient live load). The way in which the crowd is monitored varies throughout the season as different methods prove to be inadequate.

The initial plan was to use a video camera to monitor the upper deck, recording during the entirety of the game. The problem associated with the video camera was the difficulty in synchronizing the time histories to the video footage due to the stoppage time required to switch video tapes. Additionally, the video camera does not deal well with the glare due to the stadium lights during a night game, making it difficult to get a clear picture of the crowd. To counteract the glare issues, and realizing that a complete video is not necessary, still photographs were taken at ten minute intervals to give an idea of the upper deck’s occupancy level. The time signatures of the pictures are easily matched to the time histories allowing each ten minute time history to be tagged with a corresponding crowd density and dispersion.
The “game information” notes are a correlation between the events in the stadium and the corresponding measurement time histories. To better capture the nature of the loading causing certain responses, a team member monitors the response histories in real time and records what events during the game correspond to that loading. During later examination, each peak in the time history is labeled with the event that corresponds to it. Many of the peaks recorded are not caused by the rhythmic loading associated with music but are reactionary loads in response to exciting plays on the field and it is important to understand what types of events result in large vibrations, how long those vibrations last, and the crowd perception.
CHAPTER FOUR
DATA PROCESSING

The processing of the data obtained from the home games of the 2007 Clemson University football season occurs in three different phases. The first phase is the creation of an initial finite element model (FEM) using a structural analysis program. The next phase is the extraction of modal properties from the acceleration time histories. Finally, parameters used to create the initial FEM are updated such that the modal properties of the FEM more closely resemble the modal characteristics measured for the stadium.

4.1: Girder Finite Element Model Creation

The initial FEM is created using the structural analysis program SAP2000 version 10 (SAP2000). To create the model, construction drawings of the south upper deck of Clemson Memorial Stadium were obtained from the Clemson University Facilities Department (Enwright, 1979). The centerline geometry of the girder is the first information used in the creation of the FEM with a node being placed at any cross sectional transition, location of support, or location of an instrument.

The precast girders used to construct the stadium are non-prismatic members. For ease of analysis, the girders are discretized into smaller elements and treated as prismatic members using an average cross-sectional area and average moment of inertia. Because the girder is being treated as a plane frame, the resulting cross sectional properties
calculated for one girder are doubled to account for the double girder. To validate this assumption, the girders must have enough continuity that they move as though they are one piece. Inspection of the diaphragms cast between the precast girders shows no cracks supporting this assumption. Information about the torsional rigidity and weak axis properties are not needed due to the treatment as a plane frame system. The self-weight of the girders is calculated using 150 pounds per cubic foot as the unit weight of the concrete and the average cross sectional area to determine volume. The modulus of elasticity associated with the concrete is based on the concrete specified design strength of 5000 psi and calculated using the ACI 318-08 recommended equation:

\[ E_c = (w^{1.5})(33)(\sqrt{f'_c}) = (150 \text{pcf})^{1.5}(33)\sqrt{5000 \text{psi}} = 4287 \text{ksi} \]

Due to the construction methods used to construct the stadium, construction joints are present in two locations where the precast girders are spliced together. To ensure that the girder acts as a continuous member, tilt meters were installed on either side of one of the construction joints. The relative tilt on either side of the joint should be the same if joint continuity is maintained during loading. Figure 4.2 shows a comparison of the two tilt meters (instrument 10 and 11) for a short period of time during peak loading. As evident from the plot, both sides of the joint move together, with relative motion being less than ten percent for peak rotations. The small differences in relative rotation are to be expected due to the spatial offset between the two instruments.
Figure 4.1: Girder Instrumentation

Figure 4.2: Construction Joint Relative Tilt
Additionally, any noise in the system would account for a small amount of difference. Due to the small relative rotations, the girders are modeled as continuous in the FEM.

The boundary conditions of the model are of great importance when modeling the girder. The flexibility of the substructure frame supporting the main girders will dictate to what extent the frame needs to be modeled. A stiff frame allows the tops of the columns to be modeled as stationary points, or in a worst case, springs. On the other hand, a flexible frame will require the entire substructure to be modeled, adding a tremendous degree of uncertainty to the model. The additional uncertainty will also complicate the model optimization and updating task. In addition to the motion of the substructure itself, the relative motion between the girder bearing and the top of the column is also of interest when developing appropriate boundary conditions. To evaluate the boundary conditions, the acceleration time histories recorded throughout the season are reviewed to determine

Figure 4.3: Stadium Section (Coleman 1980)
the amount of motion occurring at girder bearing.

The construction drawings depict an unclear degree of fixity between the top of the column and the girder bearing. On the one hand, a neoprene bearing pad is provided indicating the expectation of some differential motion at that joint. On the other hand, a mechanical attachment and concrete “tongue” is detailed indicating the desire for a large amount of fixity. To accurately model this area, the time histories of accelerometer five and six are compared. Figure 4.5 shows a portion of a time history from the Boston College game on November 17th, 2007 during the playing of Zombie Nation’s song. As shown in the figure, the two locations move together signifying a large amount of fixity between the two locations. Figure 4.5 indicates that any movement at the girder bearing is due to deflections occurring within the substructure and not due to sliding at bearing.

![Girder Bearing Detail (Coleman 1980)](image)

Figure 4.4: Girder Bearing Detail (Coleman 1980)
Figure 4.5 shows that a small amount of horizontal motion is present at the girder bearing. In the most accurate of models, the boundary conditions should properly mimic this horizontal motion. For the purposes of this research, the amount of motion at this location must be large enough to justify the added uncertainty in the model that would be added to accurately portray the entire substructure. Figure 4.6 is a comparison of the motion occurring at the top of the column to the horizontal motion occurring at instrument one. Comparing the relative maximums experienced at each location provides insight into the degree to which the substructure is moving. The largest motion experienced at the top of the column is in the range of 8 milli-g’s as compared to over 20 milli-g’s seen at instrument one. If accelerations of similar magnitudes occurred at both locations, the entire substructure would need to be modeled to accurately capture the motion due to substructure motion. Being that the maximum motion occurring at the
bearing location is relatively small, the amount of uncertainty added to the model by modeling the substructure, outweighs the added benefit of modeling it. Even though the entire substructure is not modeled, the flexibility of the substructure should be accounted for to avoid having an overly stiff model.

As a compromise between modeling the complete substructure and using a restrained boundary condition, the substructure flexibility is included in the model as a set of linear springs and rotational springs. The rotational spring is used to model the rotational fixity due to the connection between the girder and the top of the column. The spring stiffness is initially estimated as 50% of the fixed-fixed rotational stiffness of the column. The column length used for determining the rotational stiffness is taken as the distance from

Figure 4.6: Horizontal Motion Comparison of Instrument Six and One.
the top of the column to the next horizontal bracing location. For column B, the rotational stiffness is impacted by the rotational stiffness of the horizontal member located just below the top of the column. For this condition, the fixed-fixed stiffness of both the column and the horizontal member add to the rotational stiffness. These values are only initial starting points for the model optimization and are expected to change.

For the linear springs, a bit more complicated approach is taken to estimate the lateral stiffness of the substructure. For a single-story portal frame, the stiffness of each column is only dependent on the stiffness of the individual column and its bearing condition. For the stadium substructure, there is a great deal of interaction between each of the columns making the lateral stiffness of each column somewhat dependent on the stiffness of entire structure. Figures 4.7 and 4.8 show graphically how the approximation is conducted.

The substructure is approximated as a set of springs in series. The value for each of the springs is calculated by locking all of the DOF’s and giving one DOF a unit displacement. The resulting reaction at the displaced DOF is dependent on the spring stiffness. The reaction occurring at each DOF due to a unit displacement is given as:

\[ \text{Rxn DOF 3} = K_{\text{top}} \times \text{Unit Disp.} \]
\[ \text{Rxn DOF 2} = K_{\text{top}} \times \text{Unit Disp.} + K_{\text{mid}} \times \text{Unit Disp.} \]
\[ \text{Rxn DOF 3} = K_{\text{mid}} \times \text{Unit Disp.} + K_{\text{bot}} \times \text{Unit Disp.} \]
The top of the shearwall is used as the fixed reference point (ground). Solving this series of equations, the values of each of the springs is obtained.

The girder model is created as a line model. The lines represent the neutral axis of each beam element. Due to the large depth of the girder, a substantial distance is present between the neutral axis and the top of column. Between these locations is a rigid piece of concrete. To make it so the girder node rotates about the bearing node, a body constraint is added between the two nodes such that it appears a rigid link has been
placed between the two. The constraint makes certain that the girder node rotates around the bearing node as the physical member is expected to behave.

The modeling of the geometry and properties of the girder provides an appropriate mechanism of assembling the stiffness matrix for use in modal analysis. In addition to the stiffness matrix, an accurate mass matrix is needed. To do this, the mass lumping feature of SAP2000 is utilized. The feature takes mass from both the self-weight of the structure and the applied loads and lumps them at nodes based on tributary length. The mass due to self weight of the girder is as discussed previously. The other sources of mass in the structure are broken down into: z-slabs and seats, precast handrails along the top and the bottom of the stands, the portal system, and any crowd loading. Since the initial model is created for an empty stadium condition, the crowd loading will be discussed later.

The z-slab self-weight is based on an assumed normal weight concrete with a unit weight of 150 pcf. The cross section of the z-slabs is constant for its entire span with the span increasing slightly the further from the field it is located. The weight of the z-slab was transformed into a line loading placed on the girder model. The line load takes the form of a trapezoidal shape due to the increasing span length of the z-slab. The load is treated as a uniform load applied along the girder which is a simplification of the stepped nature of the actual bearing condition. A 2 psf load is superimposed to account for the aluminum seats and the handrails installed onto the z-slab.
To model the precast handrails at the top and bottom of the stadium, the weight of each is calculated and the connection details are examined to determine an appropriate loading scenario. For the upper handrail, a simple, 5’-4” x 0’-6”, rectangular concrete member spans between posts connected to the tips of the girders (Figure 4.11). The handrail is connected at two locations on the post using simple connections. The weight associated with the handrail is divided equally between the two connection points on the post.

The bottom handrail is a precast, simply supported, concrete member (Figure 4.9) that spans between the bottom tips of the girders. The handrail called out in the construction drawings is drastically different from the “as-built” handrail. “As-built” drawings could not be located and the manner in which the handrail was installed made it difficult to estimate the size of the installed handrail. For modeling purposes, the reactions due to the handrail are determined using the cross section provided in the construction drawings. While this assumption imparts significant error into the model, this error will be accounted for in the optimization portion of the procedure. The final mass contributor is the dead load due to the access portal located next to the girder (Figure 4.10). The portal imparts its load onto the girder at one location near the bottom tip of the girder. One wall of the portal is built off of the girder using cast in place concrete and dowel rods to attach the wall to the precast girder. This load is treated as a uniform load applied to the girder. Through inspection of the connection details, it is evident that little effort was made to cast the new concrete so that it would act integrally with the precast girder. The lack of continuity at the cold joint means that additional stiffness at this location is
negligible. The opposing portal wall is a single concrete member that spans between two beams running normal to the girder. Also spanning between those two beams is the portal slab which provides the walkway from the concourse into the stands. Using basic statics and an assumed 150 pcf unit weight, the load from the portal and the portal slab is determined with the subsequent reaction onto the girder being determined. A complete description of the geometry, constraints, loads, and elements used in the initial model can be found in the appendix.
Figure 4.9: Bottom Precast Handrail

Figure 4.10: Girder at Portal

Figure 4.11: Top Precast Handrail
4.2: Modal Parameter Extraction

With the initial structural model complete, the modal properties of the actual stadium must be determined. One possible means of modal parameter identification involves forced vibration. Due to limited access, budget, equipment, and having such a large structure, this technique is ruled out. Instead, an output-only system identification technique is selected. As mentioned in Chapter 2, a number of methods for modal
parameter extraction for an output-only system are available. Due to its ease of use and effective nature, the Enhanced Frequency Domain Decomposition Method (FDD) is selected for modal property extraction. The FDD method allows for the extraction of natural frequencies, mode shapes and an estimate of the modal damping ratio.

The FDD method (Brinker et al 2001) is an extension of the classical Peak Picking Method (PP). Expressing the relationship between unknown inputs $x(t)$ and measured responses $y(t)$ as

$$G_{yy}(j\omega) = \overline{H}(j\omega)G_{xx}(j\omega)H(j\omega)^T$$  \hspace{1cm} (4.1)

where $G_{xx}(j\omega)$ and $G_{yy}(j\omega)$ are the power spectral densities of the input and the output, respectively, and $H(j\omega)$ is the frequency response function (FRF) with the overbar and the superscript $T$ indicating the mathematical operation complex conjugate and transpose, respectively. Rewriting the equation in pole-residual form, applying some mathematical manipulation, and making the observation that the PSD for the assumed input of Gaussian white noise is a constant, i.e. $G_{xx}(j\omega)=c$, equation (4.1) takes the form

$$G_{yy}(j\omega) = \sum_{k \in \text{Sub}(\omega)} \frac{d_k \Phi_k^T}{j\omega - \lambda_k} + \frac{\overline{d_k \Phi_k^T}}{j\omega - \overline{\lambda}_k}$$  \hspace{1cm} (4.2)

where $\Phi_k$ is the $k^{\text{th}}$ mode shape vector, $d_k$ is a scalar constant, $\lambda_k$ is the $k^{\text{th}}$ pole (natural frequency), and $\text{Sub}(\omega)$ is the subset of frequencies that will maximize the equation. For lightly damped structures, Equation (4.2) is the response spectral density. The next step is what sets the FDD method apart from the PP method. Once the output PSD at discrete frequencies is known, it is decomposed by singular value decomposition (SVD) giving
where $U_i$ contains the singular vectors and $S_i$ contains singular values for the SVD at the $i^{th}$ frequency. When a single mode dominates the response, the first singular vector, $u_{i1}$, is the mode shape associated with that frequency. Each singular value represents a single degree of freedom (SDOF) system at that particular frequency. Isolating each portion of the auto-PSD function around a peak, the natural frequency and damping can be determined. To do this, the portion of the SDOF PSD is taken back to the time domain using an inverse fourier transform and damping is obtained by logarithmic decrement.

The implementation of the FDD occurs in two steps: algorithm verification followed by algorithm implementation. To verify and examine the sensitivity of the FDD method, acceleration time histories obtained using the initial F.E. model subjected to a Gaussian white noise loading are examined. But first, the acceleration time histories must be created. To create the time histories, SAP2000 is used. The nature of the loading imparted on the analytical model is extremely important. An important recognition is the difference between white noise and Gaussian white noise. Simple white noise is created using a random number generator such that the value taken for any number in the string has equal likelihood. For Gaussian white noise, each number in the string is selected from a Gaussian normal curve. While both strings of numbers provide a broadband frequency content, their autocorrelation functions are much different. To check for Gaussian white noise, the autocorrelation function should closely resemble the dirac delta
function with the time lag equal to zero. Using MATLAB software, a time history consisting of Gaussian white noise is generated for use as a loading.

Two analyses are run using SAP2000 to provide a verification of the FDD method: a modal analysis and a linear time-history analysis. The modal analysis provides the natural frequencies and mode shapes associated with the model which will be compared to the modal properties obtained by the FDD method. A linear time-history analysis is used to create a 120-second long acceleration time history at a sampling rate of 200 Hz. The short length of the file (two minutes rather than the ten minute files collected from stadium testing) is due to limitations in SAP2000. Although no published limitations could be found, SAP2000 was unable to finish a more lengthy analysis. An assumed damping of 5% for all modes is used. Anticipated levels of structural damping in the stadium are lower than 5% but a higher level value is used to test the robustness of the algorithm. With the addition of the crowd into the stadium, damping levels may increase such that the FDD’s sensitivity to high damping ratios may become an issue. In anticipation of possible high levels of damping, the algorithm verification is run at a “worst case” scenario to provide insight into the possible results.

In an attempt to closely replicate the procedure to be used for the data collected from the stadium, nodes occurring at the same location as vertical accelerometers (instruments 0, 4, 7, 9) are considered. Figure 4.13 shows the singular values associated with the frequency domain for all four of the channels.
There are a number of parameters during the implementation of the FDD method that impact the algorithm. The first is the length of the fast fourier transform (NFFT) used in creating the cross-power spectral matrix. The NFFT determines the frequencies at which the power spectral density is calculated. Larger values of NFFT will decrease the frequency step increasing the precision. Lower values of NFFT will provide a smoother singular value plot making for easier peak picking. For analysis of the girder, an NFFT value of $2^{12}$ is used. This value provides a small enough frequency step while providing a smoother plot for easier peak picking. The cross power spectral density is also dependent on the windowing function used and the amount of overlap between windows. A hanning window with a $2^{11}$ point overlap is selected. The value of $2^{11}$ corresponds to an overlap of 50%.
Using a peak detection algorithm, the singular value plot is processed to identify all of the possible natural frequencies. The sensitivity of the peak detection algorithm can be adjusted to capture the desired number of natural frequencies. The peaks associated with natural frequencies are visually obvious; the use of the peak detection algorithm will make for automated data processing in later stages. The points of interest from Figure 4.13 are summarized in Table 4.1. The maximum percent difference of 1.69% is well within expectations for a system with 5% damping. As the damping ratio approaches zero, the results of the algorithm approach exact identification.

<table>
<thead>
<tr>
<th></th>
<th>f1 [Hz]</th>
<th>f2 [Hz]</th>
<th>f3 [Hz]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analytical Response</td>
<td>2.97</td>
<td>5.85</td>
<td>10.83</td>
</tr>
<tr>
<td>FDD Method</td>
<td>3.02</td>
<td>5.81</td>
<td>10.79</td>
</tr>
<tr>
<td>Percent Difference</td>
<td>1.69%</td>
<td>0.68%</td>
<td>0.37%</td>
</tr>
</tbody>
</table>

The singular vectors determined by singular value decomposition provide the mode shapes associated with the natural frequencies. Because non-proportional viscous damping of 5% was assumed for all modes, the mode shapes obtained are complex modes. The physical meaning of a complex mode shape is not as easily understood as real mode shapes. A complex mode shape means that points along the structure are no longer moving either in-phase or out-of-phase. As a consequence of the variable phase angle, points cross the equilibrium position at different times. Most FEM software packages do not include damping when calculating the frequencies and mode shapes of
the structure. Therefore, it becomes the task of the analyst to approximate “real” mode shapes from the complex ones so that one is comparing equivalent shapes. One common method of obtaining real modes for lightly damped structures is to multiply the modulus of each element of the vector by the sign of the cosine of its phase angle (Friswell and Mottershead 1995). By doing this, an element with a phase angle between -90° and 90° will be positive. This procedure is only valid for lightly damped structures; a common assumption for large civil structures.

As an example of the accuracy possible in detecting mode shapes, Table 4.2 provides the mode shapes for the first two natural frequencies of the girder model. The extracted mode shapes are nearly an exact match to the analytically determined mode shapes for the first two modes. The modal assurance criterion (MAC) values calculated are 1.000 for the first mode and 0.999 for the second. While the accuracy of the mode shape extraction is comforting, the actual shapes extracted are a bit unusual.

The girder system is unique in that the first and second fundamental modes are isolated modes. The dominant amount of motion for the first mode occurs at the upper tip of the cantilever while the second mode is dominated by the motion at the bottom tip of the cantilever. This phenomenon limits the usefulness of the extracted mode shapes. System identification with instruments 0 or 9 included will be dominated by the response of the first or second modes, making identification of higher modes difficult. Figure 4.13 is a good example of this consequence. The response from the third mode is hard to notice
due to the overwhelming power given to the first and second modes. To extract higher frequency mode shapes, the FDD method is used with the acceleration time histories from instruments 4 and 7 only. Figure 4.14 shows the frequency content of the girder based on the model nodes at the locations corresponding to instruments 4 and 7. Without the contribution of the nodes at the girder tips, the frequency associated with the third natural frequency is more easily identified.

This method works well in determining the natural frequencies of higher modes, but it lacks the ability to deliver complete mode shapes. Because only two of the nodes are used in the identification, the singular vector provides information on only those nodes. In an attempt to get a four node mode shape, the natural frequency found using Figure 4.13 is used as a marker. After creating the power spectral density matrix for all four nodes, the singular vector associated with the natural frequency previously determined is extracted. The mode shape extracted using this method is inconsistent with the expected shape provided from SAP2000.
Table 4.2: FDD Verification Mode Shape Extraction Results

<table>
<thead>
<tr>
<th></th>
<th>FDD</th>
<th>Exact</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>First Mode</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-0.0221</td>
<td>-0.0245</td>
</tr>
<tr>
<td></td>
<td>-0.0049</td>
<td>-0.0043</td>
</tr>
<tr>
<td></td>
<td>-0.0995</td>
<td>-0.0988</td>
</tr>
<tr>
<td></td>
<td>1.0000</td>
<td>1.0000</td>
</tr>
</tbody>
</table>

![First Natural Frequency Mode Shape Comparison](image1.png)

<table>
<thead>
<tr>
<th></th>
<th>FDD</th>
<th>Exact</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Second Mode</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-1.0000</td>
<td>-1.0000</td>
</tr>
<tr>
<td></td>
<td>0.2726</td>
<td>0.2754</td>
</tr>
<tr>
<td></td>
<td>0.0096</td>
<td>0.0114</td>
</tr>
<tr>
<td></td>
<td>-0.0350</td>
<td>-0.0443</td>
</tr>
</tbody>
</table>

![Second Natural Frequency Mode Shape Comparison](image2.png)
A tremendous advantage of the FDD method over the PP method is its ability to extract damping ratios. Damping is a notoriously difficult parameter to identify. The amount of damping can be dependent on non-structural elements, construction techniques, tolerances, friction, magnitude of loading, and weather, to name a few. All of these factors move the actual damping of a structure away from viscous damping, the assumed model, and adds complexity. Adding to the difficulty of obtaining reliable damping coefficients is the large scatter associated with the various techniques in calculating damping (SAMCO 2006). In a series of tests done on Braga Stadium, the variation of damping coefficients obtained by various methods is as much as 100% (Magalhaes et al
Ambient excitation techniques often overestimate the damping coefficients due to the presence of aeroelastic damping of flexible structures.

To test the reliability of the damping ratios found using the FDD method, the acceleration time histories developed in SAP2000 are once again used. As previously done, the autospectra density functions formed by the singular values of the spectrum matrix are created. The peaks around a natural frequency are then isolated, zero-padded, and brought back into the time-domain via an inverse fast fourier transform (IFFT) creating the auto-correlation functions of the SDOF system associated with that mode. The auto-correlation function takes the form of a free response of a damped SDOF system allowing for damping ratios to be determined by exponential decay. Figure 4.15 is the autocorrelation function for the first mode. The SAP2000 model was created with 5% damping in all of the modes. Applying logarithmic decrement techniques to this function, we see arrive at a damping ratio of 3.9%. This value has a 22% error associated with it.

To test if the damping was too high for the algorithm (one assumption of the FDD is low damping), acceleration time histories are created using SAP2000 without any damping. If the high damping was to blame, the algorithm should provide accurate results for the undamped structure. Figure 4.16 shows the autocorrelation function for the undamped structure. Applying logarithmic decrement techniques a damping ratio of 0.87% is
Figure 4.15: Autocorrelation function of the first mode used to determine damping

Figure 4.16: Autocorrelation function of undamped structure
determined. This rules-out too high of damping as a means of the inaccuracy. The leakage due to the approximate method used is determined to be the cause.

Although the damping coefficient for an empty stadium is not able to be accurately determined, it is not of critical concern. The updated model will be used in analyzing how the structure behaves when exposed to various types of crowd loadings. The crowd in a stadium does not only provide a means of excitation but also is a tremendous source of damping. The structural damping is combined with the damping due to the crowd. Thus, the amount of damping present in the stadium will ultimately be dictated by the size, dispersal, and activity level of the crowd. To accurately predict the level of damping present at any point in time, a full investigation of human-structure interaction is warranted. In lieu of investigating Clemson Memorial Stadium, recommendations and findings in literature are used.

Occupied damping ratios between 2 and 3% were found for Camp Randall stadium at the University of Wisconsin (Tuan 2004). These findings are based on the exponential decay of a response time history. This technique gives a rough estimate of the composite damping but does not shed light into the modal damping associated with each mode. Elland Road Stadium in the United Kingdom was investigated in detail to determine how the crowd affected the frequency and the damping of the stadium (Reynolds and Pavic, 2006). Using FDD, the team reported natural frequencies and damping ratios for six different crowd configurations:
1. Empty Structure
2. Stadium filling up with people before match
3. Stadium emptying after match
4. Half-time (people milling about)
5. People seated during match
6. People standing following goal event.

The damping findings are summarized in Figure 4.17. Average damping values for the first mode range from 2.4% to nearly 5% depending on the nature of the crowd. Upon successful updating of the model, damping values in agreement with these published values will be used in testing future loadings. Other studies of human-structure interaction are mentioned in Reynolds and Pavic but have not been verified by this work.

Figure 4.17: Damping for Various Crowd Configurations (Reynolds and Pavic, 2006)
The outcome of the verification provides a course of action and a level of expectation for the use of the FDD method for Clemson Memorial Stadium. Due to the unique nature of the first and second mode shapes, and the inability to extract quality mode shapes for the third mode, the mode shapes will be used only as a post analysis justification for both the system identification and the model optimization. The ability to determine reliable natural frequencies has been shown. The natural frequencies will be used as the objective function during the optimization process. After successful verification, the FDD method is ready for use on the real structure.

The goal of the system identification of the real stadium is to determine natural frequencies and mode shapes for the stadium at various occupancies. Initially, the stadium will be analyzed at an empty occupancy. For an empty occupancy, ambient vibrations due to wind are used to excite the stadium. Wind loadings are assumed to closely mimic Gaussian white noise (Brinker et al, 2001). The limitation of this method is that it is largely weather dependent. To get strong readings, strong sustained winds are required. Other ambient vibration studies traditionally use human or vehicular motion as an excitation source. Typically, the added mass due to the people or vehicles is proportionally small. In the case of a football stadium, a full stadium adds a significant proportion of mass to the system; mass that is not easily estimated. Being unable to ignore the additional mass, human source excitation is not able to be used.
Wind loading is used as the excitation source for tall buildings with good success in a number of studies (Ventura et al 2005). Being a tall, slender structure, weaker winds are adequate for causing motion of the building. Furthermore, the mode shapes being generated are global mode shapes as opposed to the local mode shapes being captured in this study, further reducing the wind intensity needed. While there have been a number of successes using wind based ambient vibration, the difficulties of relying on wind as an excitation source have been well documented (Brownjohn 2003). For this study, having the sensors continuously installed in the stadium and easily accessible allowed flexibility in the days data was collected. Typical applications of system identification do not allow for such flexibility.

Strong winds were recorded on August 22nd, 2008 with sustained winds of 10 mph with gusts up to 20 mph coming from the northeast. Having adequately strong winds, seventeen-ten minute acceleration time histories were recorded. Using the FDD method, as described previously, the empty stadium acceleration time histories are analyzed to obtain natural frequencies and mode shapes.

Using the previously installed instrumentation and a four-channel, 24-bit A/D convertor, seventeen- 10-minute acceleration time histories were recorded. Using the more sensitive data collection equipment allows for the use of four instruments at a time. The instruments selected were instruments: 0, 4, 7, and 9 as described in Figure 4.1. The SVD of each of the 10 minute files, do not provide consistent results as shown in Figure
4.18. This is due to the stiff structure and low levels of excitation. To extract reliable natural frequencies, the power associated with each frequency for each file is summed to form a single plot. By doing this, the peaks that are consistently present in each file will become apparent. The downside to doing this is that mode shapes cannot be determined. As mode shapes are not used as part of the objective function for model updating, they are not needed for the purposes of this research. Figure 4.19 is the combination of the singular values for each file. By doing this, the first frequency of 3.22 Hz and second frequency of 6.68 Hz. can be clearly seen.

Figure 4.18: Comparison of Individual File PSD’s for Empty Stadium
Figure 4.19: Combined Results of all 17 time history files from empty testing

The PSDs created to make Figures 4.18 and 4.19 include instruments 0 and 9, the instruments at the tips of the top and bottom cantilevers. The majority of the motion in the first and second modes occurs at these two locations making the power associated with these two instruments overwhelming compared to the contribution from the 3\textsuperscript{rd} mode. Figure 4.19 shows no indication of the presence of a higher mode within the frequency range tested. But as was shown in the algorithm verification phase of this research, it is likely that this frequency will only be present when the motion of instruments 0 and 9 is not considered when creating the PSD’s. To identify the frequency of the 3\textsuperscript{rd} mode, a the FDD algorithm is run once more, this time only considering instruments 4 and 7. Instruments 4 and 7 are located at points on the girder where motion is expected to be high for the 3\textsuperscript{rd} mode and should have enough power to identify that
mode. Using the same compiling technique as earlier, Figure 4.20 shows the results using only instruments 4 and 7.

![Figure 4.20: 3rd Mode Identification using Instruments 4 and 7](image)

Even though the motion occurring at instruments 4 and 7 due to the first and second modes is less significant, they are still prominent in the figure. Looking to higher frequency modes, the next peak visible is 10.30 Hz. Comparing this mode to the frequencies in the initial FEM, we expect to see the 3rd mode in this area. The energy required to activate this mode is higher than the energy required to activate the first two modes due to the high amount of curvature and the higher stiffness (continuous girder compared to cantilever girder).
The modes associated with the local girder bending can be determined by investigating the vertical instruments. Any global rocking of the stadium will not be identified with the vertical instruments. Investigation of the horizontal instruments throughout the season shows a consistent frequency around 2 Hz. The frequency found in each file is dependent on the size of the crowd (additional mass) but was consistently between 2.05 and 2.15 Hz. Empty stadium data was only collected for the four vertical instruments. Therefore, data from football is used to identify the global swaying of the frame. Figure 4.21 is the result from the Furman University game on September 15th, 2007. A clear peak can be identified at 2.10 Hz. The peaks of the other frequencies are not valid because of the addition of spectators mass. They are much lower than identified for an empty stadium, which is consistent with the theory of structural dynamics.

Figure 4.21: Frequency Content of Horizontal Instruments-Furman Game
The results show that there is distinguishable amount of horizontal motion present in the structure. This verifies the earlier assumption to include horizontal springs in the initial FEM for the substructure. As a means to verify the validity of the updated model, the frequency content of the model for the mode associated with global rocking of the frame should be higher than the frequency determined. This is due to the difference in mass between the two estimates.

In a similar manner to Tuan (2004), the effective damping of the crowd and structure is estimated by looking at the free response of the time histories after a harmonic loading. Figure 4.22 provides insight into the effective damping due to the crowd and structure. This time history is of instrument 0 and is selected because the time at the end of the song is accurately known. The initial post-song damping is tremendous at 10.1% of critical. The second damping estimate shows a ratio of 3.5% with the oscillations occurring at 3.6 Hz. Both estimates show a large amount of damping is present for a crowded stadium.
4.3: Finite Element Model Updating

Having obtained natural frequencies to use in an objective function for model updating, the initial model is updated such that the frequencies of the model more closely match the frequencies determined experimentally. As mentioned in Chapter 2, the process of model updating has three steps: 1) selection of an objective function, 2) selection of updating parameters, 3) implementation. Due to its ability to handle more parameters than measurements, and the ability to provide physically meaningful results, the sensitivity based model updating technique presented in Friswell and Mottershead (1995) is selected.

Figure 4.22: Free vibration after harmonic crowd loading
From the system identification portion of this research, three natural frequencies were identified for use in model updating. The frequency associated with the side sway of the frame cannot be used as an updating criterion because of the lack of substructure mass in the model. This gives the frequencies of 3.22, 6.68, and 10.30 Hz as the target frequencies, $\lambda_{ei}$, in the objective function:

$$\Pi_1 = \sum m \alpha_m \left( \frac{\lambda_{am} - \lambda_{em}}{\lambda_{em}} \right)^2$$

(4.4)

where $\lambda_{em}$ is the $m^{th}$ experimentally determined natural frequency, $\lambda_{am}$ is the $m^{th}$ analytical natural frequency, and the weighting factor, $\alpha_m$, set as one for all three frequencies, giving equal weight to each frequency in the final response. Due to the limited number of objective criteria and the isolation of the first two modes, it is important that the model converges to all three frequencies.

The selection of the updating parameters is a crucial step in achieving the best results possible. The process of selecting a manageable number of updating parameters from all of the possible parameters is defined in detail in Chapter 2. Using the guidance set forth in Chapter 2, a set of updating parameters is selected.

The parameters selected to be updated can be set into two categories; parameters that relate to the stiffness matrix and those that relate to the mass matrix. The parameters impacting the mass matrix are first addressed. The initial FEM is created using an assumed weight of concrete of 150 pcf and dimensions dictated in the construction
drawings. From this information, the weight of the z-slabs, girders, portal, and handrails are estimated. Each of the weights calculated have some amount of expected error and are therefore candidates for updating. To limit the number of parameters in the updating procedure, the mass of the z-slabs and girders are combined into one parameter since they are always present together and are global along the entire length of the girder. All three of the frequencies are sensitive to the z-slab and girder weights. The handrails at the top and bottom of the stands will impact the first and second frequency, respectively. The bottom handrail has a large degree of uncertainty because the handrail installed in the stadium appears to be different than the handrail in the construction drawings. The final source of mass in the model is a portal installed adjacent to the girder in the area of column A. Part of the portal load is applied in the same area as the front handrail with the other load being applied by beam UDB5, as indicated in the stadium plans. The uncertainly associated with both of these numbers is extremely high. The mass of the portal will directly impact the 2nd and 3rd modes. The mass of the system is lumped into four updating parameters, each having an impact on one or more frequencies and containing error.

The stiffness of the system comes from two places, the girders and the boundary conditions. The stiffness of the girders is dependent on three major parameters: cross-sectional area (A), moment of inertia (I), and modulus of elasticity of the concrete (E). All three of the frequencies are sensitive to the moment of inertia of the girders and the modulus of elasticity of the concrete while the cross-sectional area of the girders is found
to have little impact on the frequency content of the structure and is therefore excluded as an updating parameter. The discretization of the girders into prismatic sections based on an average moment of inertia will introduce some uncertainty into the values used. Furthermore, the moment of inertia is calculated for an uncracked section which is the normal assumption for post-tensioned members but may not be completely accurate. Possibly the most uncertain value used in the initial model creation, the modulus of the concrete is included as an updating parameter. All of the frequencies are sensitive to this parameter and a tremendous amount of uncertainty exists in the initial value. Using the design strength specified in the construction drawings and the recommendations of ACI for calculating the modulus, an initial value is established. While the design strength of concrete is specified, the delivered strength of concrete is unknown, directly influencing the actual modulus. Furthermore, the ACI recommendations are for the static modulus of elasticity. The behavior of concrete is slightly different for small strain, short duration loading and is classified by a dynamic modulus of elasticity. The dynamic modulus of elasticity has been found to be roughly 25% higher than the static modulus (Han and Kim 2004). All of these factors provide a large amount of uncertainty associated with the modulus of the concrete. Each of these parameters may have a different value in each of the elements in the model and could be treated as separate parameters for each element. By doing this, the number of parameters needed to be updated would increase from 3 to well over 50. Having that many parameters to update would provide little in meaningful results and require enormous amount of computing time. In an effort to minimize the
number of updating parameters, A, E, and I are treated as global parameters with the same modifying coefficient being applied to all of the elements.

The boundary conditions of the model greatly impact the frequency content of the model and should be included in the consideration of updating parameters. The translational springs are found to have little impact on the frequency content of the model and are not considered as updating parameters. The rotational springs, on the other hand, can impact the frequency content of the model and have a great deal of uncertainty associated with their initial value. The word “can” is used to indicate an interesting phenomenon with boundary condition springs. The rotational springs can range between two extreme values, a fixed condition (infinite stiffness) and a pinned condition (zero stiffness). If the initial value of the spring stiffness is such that it approaches a fixed condition, small perturbations in the stiffness of the spring will result in no change in the frequency content of the structure. This is because the motion of the structure is being dictated by the girder stiffness at that location, not the boundary condition. A slight change in a fixed condition is still a fixed condition. The same can be said for a pinned condition. Figure 4.23 shows the first and second frequencies of the initial model to varying spring coefficients.
By going through the parameter selection process, the following 9 parameters are chosen for model updating: 1) global mass (girders and z-slabs), 2) top handrail, 3) bottom handrail, 4) Beam UDB5 reaction, 5) concrete modulus of elasticity, 6) girder moment of inertia, 7) rotational spring coefficient at column A, 8) rotational spring coefficient at column B, 9) rotational spring coefficient at column C. Each of these parameters will be changed through the use of a coefficient that the parameter will be multiplied by. The coefficients are used so that each parameter has equal weight numerically as required by updating method.
The updating method used in this research is a sensitivity based procedure presented in Friswell and Mottershead (1995). The method uses the previously selected objective function as the convergence criteria. Objective functions are normally non-linear functions of the parameters. Solving these functions “directly” requires the use of polynomial based optimization techniques which require the analyst to select a correct form of the relationship before it may be solved. An alternative to this practice is to linearize the relationship between the objective function and the parameters using a truncated Taylor series expansion to produce the linear approximation

$$
\Delta Z = S \Delta P
$$

This relationship states that a change in any parameter, $\Delta P$, multiplied by the sensitivity matrix, $S$, will result in a change in the analytical frequencies, $\Delta Z$. The sensitivity of each frequency to a perturbation in each parameter is calculated at each iteration and forms the sensitivity matrix, $S$. The sensitivity matrix will have the size $(m \times n)$ where $m$ is the number of measured outputs and $n$ is the number of parameters. In a theoretical sense, the sensitivity matrix is the first derivative of each frequency with respect to each parameter. To do this a functional relationship between the frequency and the parameter must be determined and its derivative taken. This procedure is complicated and computationally intensive. As an alternative method of determining the sensitivity matrix, each sensitivity can be determined as

$$
S_{m,n} = \frac{\Delta Z_m}{Z_m} \frac{P_n}{\Delta P_n}
$$
where $\Delta Z_m$ is the change in the $m^{th}$ frequency ($Z_{i+1} - Z_i$) which is divided by the $m^{th}$ frequency, and $\Delta P_n$ is the perturbation of the $n^{th}$ parameter divided by the $n^{th}$ parameter occurring at the $i^{th}$ iteration. It is the formation of this matrix during each iteration which requires a large amount of computing power because the FE model must be evaluated for every entry in the matrix.

For the case of more parameters than measurements, the optimization problem can be solved using the Lagrange multiplier technique yielding:

$$P_{i+1} = P_i + S_i^T \left[ S_i S_i^T \right]^{-1} \left( Z_m - Z_i \right)$$  \hspace{1cm} (4.7)

Introducing the parameter weighting matrix, $W_{PP}$, and the frequency weighting matrix, $W_{ZZ}$, the change in the parameter is calculated as

$$P_{i+1} = P_i + [S_i^T W_{ZZ} S_i + W_{PP}]^{-1} S_i^T W_{ZZ} \left( Z_m - Z_i \right)$$  \hspace{1cm} (4.8)

The introduction of the weighting matrices is what gives flexibility and power to this updating method. The accuracy with which the experimentally determined frequencies are measured may be different between values. It is common for lower energy modes to be measured more accurately than higher frequency modes (Friswell and Mottershead, 1995) and as a consequence lower frequency modes should have more emphasis placed on them in the optimization. A similar situation arises when the objective function has
both frequencies and mode shapes. Frequencies are normally identified to within a few percent whereas mode shapes may contain upwards of 10% error. The weighting matrix, $W_{zz}$, allows the engineer to place more weight on the measurements they feel most confidence in; thus driving the results to match these values before they converge on the other measurements. The measurement weighting matrix is a square, diagonal matrix whose values are given by the inverse of the variance in the measurement. In other words, more accurate measurements will have a higher relative value entered in the matrix than more questionable measurements. The same is true for the parameter weighting matrix, $W_{pp}$. Parameters with large uncertainties are given smaller magnitude entries than parameters that are better estimated.

The scaling of the weighting matrices will impact the speed of convergence. A low magnitude scale will move closer and closer to the convergence. If the parameter value is plotted against the iteration number, the plot would be a smooth curve with the slope becoming zero near the end of the optimization (indicating convergence). A high magnitude scale will overestimate the change in the parameter needed and will overcompensate after each iteration until convergence finally occurs. In this case, a plot of the parameter value after each iteration would have a saw tooth appearance to it. A happy medium between the two cases is desired. Low magnitude changes in the perturbation will increase the time to convergence, whereas, too high of magnitude can shoot the parameter outside of the constraints of the problem causing it to bounce back and forth between the upper bound and lower bound without convergence.
To keep physically meaningful results, the problem is constrained such that large perturbations become undesirable or are outright restricted. The first method of constraint is based on a penalty function. The penalty function, as the name indicates, penalizes the optimization whenever the parameters selected are out of the bounds set forth by the analyst. This is done by increasing the frequency residual calculated if a parameter is used to calculate that frequency that is not desirable. By using the penalty function, the final parameters selected may be out of the bounds set forth by the optimization if it can successfully minimize the objective function even with the added penalty of undesirable parameters. The other method of constraining the optimization is by outright prohibiting the parameters from going outside of the constraints. This is the process selected for this research. The constraints on this optimization are loose enough that parameters that fall outside of the bounds will no longer be defensible and are therefore not useful.

To gain confidence in the optimization procedure and a more thorough understanding of its workings, the algorithm is implemented for a cantilever beam shown in Figure 4.24.
The cantilever beam is composed of seven elements having a total of sixteen degrees of freedom (DOF). The stiffness of each element can be varied and the mass of the element is determined by tributary width mass lumping methods. To implement the algorithm, MATLAB, is utilized (Mathworks, 2008). Using randomly selected stiffness values for each element, the natural frequencies of the target structure are found by solving the eigenvalue problem. The target structure is summarized in Table 4.3.

Table 4.3: Target Parameters and Frequencies for Updating Algorithm Verification

<table>
<thead>
<tr>
<th>Entry #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>EI [N*m^2]</td>
<td>3500</td>
<td>4500</td>
<td>5500</td>
<td>5200</td>
<td>3700</td>
<td>3700</td>
<td>5300</td>
</tr>
<tr>
<td>Fn [Hz]</td>
<td>39.02</td>
<td>229.8</td>
<td>587.24</td>
<td>1064.42</td>
<td>1613.08</td>
<td>2214.92</td>
<td></td>
</tr>
</tbody>
</table>

All seven of the stiffness values are used as updating parameters and the sensitivity of the algorithm is tested by varying the number of frequencies used in the objective function. As expected, the more frequencies used in the objective function, the better the results are. As a matter of fact, the algorithm converges on faulty parameters but correct frequencies for the combination of three frequencies and seven parameters as shown in Table 4.4.
Table 4.4: Analytical Updating Results – Three Frequencies, Seven Parameters

<table>
<thead>
<tr>
<th>Entry #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>EI Initial</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
</tr>
<tr>
<td>EI Target</td>
<td>3500</td>
<td>4500</td>
<td>5500</td>
<td>5200</td>
<td>3700</td>
<td>3700</td>
<td>5300</td>
</tr>
<tr>
<td>EI Final</td>
<td>3896</td>
<td>4666</td>
<td>3955</td>
<td>4473</td>
<td>4266</td>
<td>4226</td>
<td>4446</td>
</tr>
<tr>
<td>% Difference</td>
<td>-11.3%</td>
<td>-3.7%</td>
<td>28.1%</td>
<td>14.0%</td>
<td>-15.3%</td>
<td>-14.2%</td>
<td>16.1%</td>
</tr>
<tr>
<td>Fn Target</td>
<td>39.0</td>
<td>229.8</td>
<td>587.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fn Final</td>
<td>39.3</td>
<td>229.7</td>
<td>587.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>% Difference</td>
<td>-0.71%</td>
<td>0.02%</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

When dealing with a rank deficient optimization (more parameters than measurements) an infinite number of solutions exist. This is a wonderful illustration of getting the right response for the wrong reasons. This misleading result is a function of the simplicity of the structure but it does demonstrate the limitations of this method. The algorithm converged on local minima without finding the global minima. To verify that the algorithm could produce the correct parameters, it is run with an objective function based on six frequencies. The findings are summarized in Table 4.5.

Table 4.5: Analytical Updating Results – Six Frequencies, Seven Parameters

<table>
<thead>
<tr>
<th>Entry #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>EI Initial</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
<td>4500</td>
</tr>
<tr>
<td>EI Target</td>
<td>3500</td>
<td>4500</td>
<td>5500</td>
<td>5200</td>
<td>3700</td>
<td>3700</td>
<td>5300</td>
</tr>
<tr>
<td>EI Final</td>
<td>3550</td>
<td>4525</td>
<td>5490</td>
<td>5180</td>
<td>3678</td>
<td>3682</td>
<td>5310</td>
</tr>
<tr>
<td>% Difference</td>
<td>1.40%</td>
<td>0.50%</td>
<td>0.20%</td>
<td>0.40%</td>
<td>0.60%</td>
<td>0.50%</td>
<td>0.20%</td>
</tr>
<tr>
<td>Fn Target</td>
<td>39.0</td>
<td>229.8</td>
<td>587.2</td>
<td>1064.4</td>
<td>1613.1</td>
<td>2214.9</td>
<td></td>
</tr>
<tr>
<td>Fn Final</td>
<td>39.2</td>
<td>229.8</td>
<td>587.2</td>
<td>1064.4</td>
<td>1613.1</td>
<td>2214.9</td>
<td></td>
</tr>
<tr>
<td>% Difference</td>
<td>0.40%</td>
<td>0.02%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
<td></td>
</tr>
</tbody>
</table>
By providing more frequencies in the objective function, the algorithm successfully avoids the local minima that trapped the three frequency optimization. One way to ensure that the updated result is a global minimum is to vary the initial parameters used and select the final set of parameters that make the most physical sense.

Having identified the limitations and verified the success of the algorithm, the algorithm is implemented for the stadium. The process of model updating is once again implemented using MATLAB with the aid of SAP2000. The MATLAB script can be seen in the Appendix. The process is summarized as follows.

The initial parameter values are selected and are paired with a multiplier. It is the multiplier that is actually modified throughout the algorithm. The experimentally identified frequencies are entered for use in evaluating the objective function. The weighting matrices are identified. The initial model is evaluated to determine its frequency content. The model is evaluated by creating an S2K text file that can be read by SAP2000. MATLAB then calls SAP2000, loads the file, runs a modal analysis, records the resulting natural frequencies, and closes SAP2000. Now having a starting point from which to work, each of the parameters is given an initial perturbation in order to calculate the sensitivity matrix. The identified sensitivity matrix is used to calculate the next perturbation of the parameters. If this perturbation sets the parameter value outside of the constraints, the value is brought back inside the constraints and the new model is evaluated to determine its frequency content. The process is continued for the
set number of iterations. After each step, the parameters and the corresponding natural frequencies are recorded. Finally, because the process is completely automated and takes around five minutes an iteration, an email message is sent out every ten iterations with the results so that the process may be monitored remotely. The results of the optimization can be seen in Tables 4.6 and 4.7 and Figures 4.25 through 4.29.

The implementation of the algorithm makes use of the weighting matrices previously defined. The parameter weighting matrix, $W_{PP}$, has entries of one along the diagonal except for the fifth entry (corresponding to the modulus of elasticity), which has a value of 0.7. By doing this, the modulus of elasticity is allowed more flexibility to change. The frequency weighting matrix, $W_{ZZ}$, has diagonal entries of 1.00, 0.5, and 0.25 thus giving the first frequency, which is estimated the most accurately, the most influence on the results.
<table>
<thead>
<tr>
<th>Parameter</th>
<th>units</th>
<th>Initial Value</th>
<th>Initial Coeff.</th>
<th>Upper Constraint</th>
<th>Lower Constraint</th>
<th>Final Coeff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Mass</td>
<td>multiplier</td>
<td>1</td>
<td>1.00</td>
<td>1.30</td>
<td>0.80</td>
<td>1.30</td>
</tr>
<tr>
<td>Top Handrail</td>
<td>multiplier</td>
<td>1</td>
<td>1.00</td>
<td>1.25</td>
<td>0.75</td>
<td>1.11</td>
</tr>
<tr>
<td>Bottom Handrail</td>
<td>multiplier</td>
<td>1</td>
<td>1.00</td>
<td>1.40</td>
<td>0.80</td>
<td>1.06</td>
</tr>
<tr>
<td>Beam UDB5</td>
<td>multiplier</td>
<td>1</td>
<td>1.00</td>
<td>1.40</td>
<td>0.80</td>
<td>1.08</td>
</tr>
<tr>
<td>E of Concrete</td>
<td>[ksi]</td>
<td>4287</td>
<td>1.30</td>
<td>1.50</td>
<td>1.00</td>
<td>1.08</td>
</tr>
<tr>
<td>I of Girder</td>
<td>multiplier</td>
<td>1</td>
<td>1.00</td>
<td>1.20</td>
<td>0.80</td>
<td>0.82</td>
</tr>
<tr>
<td>Spring Column A</td>
<td>[kip*ft/rad]</td>
<td>15639383</td>
<td>1.00</td>
<td>4.00</td>
<td>0.10</td>
<td>0.98</td>
</tr>
<tr>
<td>Spring Column B</td>
<td>[kip*ft/rad]</td>
<td>2646349</td>
<td>1.00</td>
<td>4.00</td>
<td>0.10</td>
<td>1.00</td>
</tr>
<tr>
<td>Spring Column C</td>
<td>[kip*ft/rad]</td>
<td>1849225</td>
<td>1.00</td>
<td>4.00</td>
<td>0.10</td>
<td>0.99</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Fn1</th>
<th>Fn2</th>
<th>Fn3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Experimental</td>
<td>3.22</td>
<td>6.68</td>
<td>10.30</td>
</tr>
<tr>
<td>Initial FEM</td>
<td>4.03</td>
<td>8.32</td>
<td>13.71</td>
</tr>
<tr>
<td>Updated FEM</td>
<td>3.22</td>
<td>6.68</td>
<td>10.29</td>
</tr>
<tr>
<td>% Difference</td>
<td>0%</td>
<td>0%</td>
<td>0.10%</td>
</tr>
</tbody>
</table>
Figure 4.25: FE Model Updating Results – Parameters 1, 2, and 3
Figure 4.26: FE Model Updating Results – Parameters 4, 5, and 6
Figure 4.27: FE Model Updating Results – Parameters 7, 8, and 9
Figure 4.28: FE Model Updating Results – Frequencies
Figure 4.29: FE Model Updating Results – Mode Shapes
The updating successfully converged on all three of the target frequencies to within a fraction of a percent. Comparing the frequencies of the initial model to the target frequencies, it is apparent that the initial model is a combination of too stiff and too light. The updating procedure is able to alter the parameters to converge on the target frequencies but this alone does not validate the outcome of the updating. To ensure that a physically meaningful model has been created, a careful review of the final parameters is done.

The modulus of elasticity increased, as one would expect, owing to concrete overstrength and to a small degree, the dynamic modulus of elasticity. Although the increase is not large, the updated modulus of elasticity would be comparable to a 6 ksi concrete or a 20% increase in concrete strength. The moment of inertia of the girders is reduced to 81%. The girders are non-prismatic members modeled as prismatic members with gross cross-sectional properties; a variance in either one of these estimations will produce a change in the moment of inertia. Although it is assumed that prestressed girders are not cracked, years of cyclic loading may have invalidated this assumption. The change in the rotational springs is insignificant. The initial values of the rotational springs based on 50% of the fixed-fixed stiffness of the columns formed a nearly fixed spring. Any small change in a nearly fixed spring results in a nearly fixed spring. The same can be said for a proportionally flexible spring. Attempts to make these parameters more active in the updating by altering their initial values are inconclusive.
The global mass of the structure (z-slab and girder self-weight) changed the most of all the parameters; increasing by 30%. Mass is dependent on a number of assumptions including the cross-section, unit weight, estimates of non-structural elements, mass lumping techniques. The combination of all of these factors may lead to a gross miscalculation of the dynamic mass and have been encountered in similar research (Jaishi and Ren, 2005). The change in the mass of the top handrail is expectedly small. Cross-sectional drawings of the handrail in the construction drawing did not show a small portion of concrete that was later identified in the field. The detail used to calculate the cross-section indicated a completely vertical inside face of the handrail. As seen in Figure 4.30, a bump-out exists about a quarter of the way down. This additional mass was not considered in the initial load calculation, explaining the 11% increase.

Figure 4.30: Top Precast Handrail
The mass of the front handrail and the portal reaction is intertwined due to the framing plan. Strong estimates of the load path were made but error is to be expected.

The mode shapes corresponding to the updated model are in line with anticipated shapes. The majority of the motion for the first mode is measured at the tip of the cantilever with little motion occurring at other locations along the girder. Mode 1 in Figure 4.29 is in agreement with these findings. In a similar manner as the first mode, experimental results indicate the second mode is isolated to the bottom tip of the cantilever. Once again, the mode shape of the updated model is in agreement with this anticipated behavior. Furthermore, based on experimental findings, the vertical motion of the third mode shape is expected to be the highest halfway between the column lines. The results of the updated model correspond to these findings.

The natural frequency representing a global swaying of the structure is determined experimentally to be 2.10 Hz. After model updating is complete, the natural frequency, consisting of the global swaying of the structure, is found to be 2.51 Hz. The values are not expected to match due to the missing mass of the substructure in the updated model. The experimentally obtained natural frequency of 2.10 Hz. includes both the mass and stiffness of the substructure, whereas the model only considers the stiffness. A system with less mass and the same stiffness is expected to have a higher natural frequency. The model is consistent with this anticipated behavior.
The results of the model updating procedure are encouraging. The algorithm successfully converged on all three of the target frequencies identified using system identification techniques. The output-only system identification technique used to determine the natural frequencies of the stadium was not without limitation. The wind was rarely strong enough to excite the structure to a magnitude large enough to overcome the noise in the system and human based excitation was not desirable due to the added mass from the crowd. Overcoming these limitations, a total of four natural frequencies are clearly identified. The true success of these efforts will be measured in the next chapter.
One application of model updating projects is to produce a model that more accurately exhibits the behavior of the structure for use in analyzing future loadings. Being able to accurately predict a structure’s behavior to abnormal loads can aid engineers in understanding the limitations of the structure. This is especially true when serviceability issues are being investigated. Most updating procedures are done for linear models and are therefore expected to be valid for loadings that stay within the linear-elastic range. Serviceability issues associated with vibrations are commonly within the linear-elastic range making them good cases to test the effectiveness of updated models at predicting structural response.

Having developed a well correlated model in the previous chapters, this chapter will test the model’s effectiveness at reproducing responses measured due to various crowd loadings. Making reasonable estimates of crowd size, distribution, and activity level, two scenarios are selected for comparison between analytical results and experimental observations:

1. Stadium at full occupancy with a portion of the crowd producing a rhythmic loading.
2. Partially filled stadium subjected to rhythmic loading.
For the first comparison, the size of the crowd is estimated and the level of crowd participation is varied until response levels match those recorded during stadium monitoring. The second comparison assumes a percentage of spectators bouncing and compares two independent judgments of percent occupancy. Each comparison should provide insight into the usefulness of updated model in predicting structural response.

5.1: Human induced rhythmic loading estimates

The coordinated bouncing of a crowd is a complicated loading to model. The loading has two major components, a harmonic portion and a stochastic portion. The harmonic portion will be dictated by the tempo of the stimulus while the stochastic portion is based on the crowd’s ability to maintain that beat. In an idealized bouncing situation, the forcing will be truly harmonic, producing a steady state response of the structure given adequate time for the transient response to damp-out. The addition of the stochastic component will continually add a transient response to the structure. This is an important observation because the transient response will add to the steady state response producing, at times, a higher response than a harmonic load will produce. Research has shown that for large crowds, the stochastic response component becomes small enough to be negligible (Parkhouse and Ewin, 2006). Therefore, the start-of-the-art is to produce a forcing function that is harmonic using the average response of a large number of trials.

One major investigation of crowd induced rhythmic loading measured the force imparted by people bouncing and bobbing (vertical motion without leaving the ground) (Parkhouse
and Ewin, 2006). Parkhouse and Ewins (2006) studied the vertical loading due to bouncing and bobbing in an effort to develop empirical loadings for groups between 5 and 200 people for tempos ranging from 90 to 210 beats per minute (bpm). Based on their findings, the crowd induced rhythmic loading can be idealized as

\[ P(t) = W(1 + \sum_{n=1}^{\infty} r_n \cos(360nf - \phi_n)) \]  

(5.1)

where \( W \) is the weight of the crowd, \( f \) is the forcing frequency given in Hertz, \( r_n \) and \( \phi_n \) are the dynamic load factor and the phase lag of the \( n^{th} \) harmonic, respectively.

Parkhouse and Ewins (2006) provide a wide range of values for the dynamic load factor based on the size of the crowd, type of motion (bouncing or bobbing), and forcing frequency. They also provide guidance on choosing appropriate phase lag factors.

Using the formerly mentioned text, an idealized load is created for use in evaluating the effectiveness of the updated model. The rhythmic loading measured throughout the season was caused by Zombie Nation’s song, *Kernkraft 400*; which has a tempo of 140 beats per minute, or 2.333 Hz. Using the tables created for a crowd of 200 spectators (the largest crowd for which values are published), dynamic load factors and phase lags for each harmonic are determined. The dynamic load factors are determined to be 1.011, 0.281, and 0.0325 for the first, second, and third harmonics, respectively. Based on literature recommendations, phase lags of 37.5, 69.5, and 88 degrees are used. The weight of the crowd is assumed to be 1 so that the function may be scaled appropriately for various applications. Substituting the values into Equation 5.1 produces the 10-
second loading function seen in Figure 5.1. This function is scaled and applied to the structure as necessary.

![Figure 5.1: Rhythmic Crowd Loading Function (Normalized to Weight of One)](image)

5.2: Full Occupancy Rhythmic Loading

The strongest response measured during vibration monitoring occurred during the Boston College football game on November 17th, 2007. Peaks over 50 milli-g’s were recorded at the tip of the cantilever due to the in-time jumping of the crowd. Figure 5.2 is a photo captured around the time of the measured response. The photo indicates that the upper deck is filled to maximum occupancy. Although the upper deck is completely filled, not
all of the spectators participate in the jumping. Utilizing the updated model and recorded time histories from the actual loading, an estimate of the level of crowd participation can be obtained.

![Figure 5.2: Full Occupancy Crowd](image)

To accurately predict the level of crowd participation, the added mass of the crowd must be included in the model. This particular section of the stadium has 29 rows with 36 seats per row except for the first 5 rows, which, due to the portal, have 8 fewer seats. This corresponds to a capacity of 1004 spectators tributary to the girder being monitored. The average weight of a spectator is presumed to be 160 pounds, based on similar work (Tuan 2004). A proportion of the crowd is assumed to be passive (not contributing to the rhythmic loading) with the remaining portion is treated as active. The weight of the
passive crowd is applied to the model as a static line load in a uniform manner. The active crowd weight is used to scale the forcing function determined previously. The forcing function is applied evenly along the length of the girder. By varying the proportion of passive to active participants, an indication of the level of participation needed to recreate the measured response can be determined.

As mentioned in Chapter 4, the damping of a crowded stadium is dependent on the size and activity of the crowd. Due to research limitations, damping ratios for Clemson Memorial Stadium are not determined for various occupancies. Instead, values determined from research on human-structure interaction are used to determine appropriate damping ratios. Using the findings of Reynolds and Pavic (2006), a damping ratio of 4.5% is assumed for the first two modes. This ratio is bounded by the two free-response damping estimates shown in Chapter 4 and is in line with Reynolds and Pavic’s findings summarized in Figure 5.3.

Figure 5.3: Damping for Various Crowd Configurations (Reynolds and Pavic, 2006)
Varying the active crowd to passive crowd ratio, the intensity of the vibrations determined using the girder model can be compared to the measured vibrations. For comparison purposes, two locations are chosen. The first is the upper cantilever tip. This location is chosen because it is the location of maximum response. The other location chosen is the vertical accelerations in the location of instrument 4 (midpoint between columns B and C). An initial estimate of 25% active produced reasonable levels of vibrations in the location of instrument 4 but vibration levels were too high at the cantilever tip. Reducing the active crowd proportion to 20% produces a good correlation between experimental and analytical results as shown in Figures 5.4 and 5.5.

![Graph](image)

Figure 5.4: Tip of Top Cantilever Response to Rhythmic Loading
Examining Figure 5.4 first, the frequency content of the two responses is similar. The experimental time history oscillates at an estimated frequency of 2.25 Hz, close to the forcing frequency. The analytical response oscillates at 2.33 Hz, the exact forcing frequency. The small discrepancy between the frequency content is due to the stochastic portion of the loading found in the experimental measurements. The major difference between the two responses is the number of modes contributing to the response. The analytical response is a smooth curve dominated by the first mode response. The experimental response is choppier due to the response of the inactive crowd milling around.

The two responses at instrument 4 are comparable in both frequency content and amplitude. The frequencies of both the experimental and analytical responses match the
forcing frequency of 2.33 Hz. Once again, the major difference between the two responses is the number of modes contributing to the response. The analytical response indicates that two modes are dominating while the experimental response indicates the presence of the inactive crowd milling around.

The comparison of the two responses indicates that the updated model has the ability to recreate the response captured during monitoring. Similar overall frequency content is seen between both responses with the major discrepancy being the small amplitude response of the passive crowd. For the purpose of estimating the effect of future loadings on the stadium, the amplitude is the most desired piece of information. The models ability to reproduce similar amplitudes for reasonable assumptions of crowd mass and activity level further validates the updated model.

5.2: Partial Occupancy Rhythmic Loading

Estimation of the full occupancy loading was more or less only dependent on the activity level of the crowd; no human judgment is needed in determining the mass and dispersion of the crowd. To recreate the situation of partial occupancy, two factors come into play: the dispersion of the crowd and the activity level of each section. The perceived occupancy based on photographs of the crowd will greatly impact the magnitude of the response determined using the model. Without going through and identifying the weight, location, and activity level of every occupant in the stadium, approximations must be made to account for the crowd. To see what impact the judgments of crowd size have on
the response of the model, two people were asked to independently give the percent occupancy of the top, middle, and bottom third of the section being monitored based on Figure 5.6. Table 5.1 provides a summary of their responses. Having viewed the same picture, each estimate of crowd size is different. The top third estimates are the most inconsistent, meaning, the response at the cantilever tip will be quite different between the two models. It is anticipated that similar responses will be seen for both analytical cases between columns B and C.

Table 5.8: Perceived Percent Occupancy of Crowd Based on Figure 5.6

<table>
<thead>
<tr>
<th></th>
<th>Bottom Third</th>
<th>Middle Third</th>
<th>Top Third</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crowd “A”</td>
<td>70%</td>
<td>80%</td>
<td>60%</td>
</tr>
<tr>
<td>Crown “B”</td>
<td>60%</td>
<td>65%</td>
<td>35%</td>
</tr>
</tbody>
</table>

Figure 5.6: Partial Occupancy Crowd
For the comparison of the two crowd sizes, the percentage of the crowd that is participating in the rhythmic loading is held constant. The participation rate is estimated based on video recordings of the crowd during the loading event. The top and middle third of the stadium have 10% of the crowd active while the bottom third has only 5% active. The low participation rates are due to the lack of the excitement surrounding the game. Additionally, the smaller crowd dictates that a different damping ratio be used. An assumed damping ratio of 4% for the first two modes is used to generate the response time histories for both cases. The passive crowd is treated as static line loads with different magnitudes depending on the degree of occupancy in each section of the girder. The active crowd is applied as a time history function using the loading function shown in Figure 5.1. The forcing function is scaled to correspond with the percent occupancy and the activity level specified.

The response of the cantilever tip due to both of the load cases is shown in Figure 5.7. As expected, the amplitude of the response of Crowd “A” is much higher than Crowd “B”. Comparing each of the responses to the measured response indicates that Crowd “B” provides a better estimate of the actual size of the crowd. This, of course, assumes that the assumed participation level is accurate for both cases. The frequency of the response for each load case matches the forcing frequency; as does the experimental response. The experimental response indicates the passive crowd contributes to the response while the model only shows one mode contributing to the response. These results confirm that a poorly defined loading will result in large errors in the results.
Comparing the response of the two crowd configurations at the mid-point between columns B and C, the two cases compare more favorably. The response due to crowd “A” is higher than the response due to “B” but both are in agreement with the experimental response. The mass assigned to this area of the structure is similar between
Figure 5.8: Instrument “4” Response (a) Crowd “A” (b) Crowd “B”
the two cases, suggesting that poor mass estimates elsewhere in the structure do not have a tremendous impact on the results at other locations.

The frequency content of both responses is the same; having been dictated by the forcing frequency. A close-up view of the frequency content of the experimental response, as seen in Figure 5.9, shows low amplitude responses contributing to the response which are not present in the analytical response. The lack of intermediate peaks in the analytical response suggest that the passive crowd may impart some vibrations.

![Figure 5.9: Instrument “4” Response Close-up – Crowd “A”](image)

Through the full occupancy and partial occupancy comparisons run using the updated model, it has been shown that the amplitude of the analytical response closely matches
the experimental results. The analytical response for the steady state lacks some of the
intermediate peaks found in the experimental work but the overall response frequency is
a match. Taking into account the approximate nature of the loading, the results shown
here validate the accuracy of the model. The results of the model are sensitive to the size,
dispersion, and activity level of the crowd used to load the model.
CHAPTER SIX

CONCLUSIONS

This work aims to develop a finite element model for use in determining the response of the south upper deck of Clemson Memorial Stadium to various loading scenarios. To achieve this goal, modal testing of the structure is conducted. Using Frequency Domain Decomposition, a total of four natural frequencies are extracted. The effectiveness of using ambient vibrations to capture acceleration time histories proved difficult. The relatively stiff structure required high levels of wind to excite the structure to a level where the response overcame the noise in the system. Even with strong winds, the results of FDD were not consistent for each file. Each 10-minute acceleration time history had to be compiled before a clear indication of the peaks could be seen. Compiling the data provided a means to identify the natural frequencies, but the mode shapes could not be accurately identified. If FDD is to be applied to similar structures for use in structural health monitoring, a method for extracting mode shapes must be developed.

For the purpose of this research, FDD provided a tremendous tool for system identification. The visual nature of the results gave the user an understanding of the relative accuracy of the results, all the while providing an immediate glimpse into the modal properties of the structure. Tall slender peaks indicated the presence of dominant frequencies with relatively small damping, whereas, lesser peaks indicate a frequency that is not excited to the same level.
The sensitivity based updating procedure is an effective tool. Limitations such as convergence to local minima and a lack of engineering judgment have been previously identified and solutions have been proposed. The validation of the tool using a simple beam showed how easily a false result can be achieved for simple structures. Using weighting matrices, a wide range of initial parameter values, and different combinations of parameters and measurements should overcome those limitations.

The updating procedure implemented herein, consisted of nine parameters and three measurements. The constraints placed on the problem were loose enough to allow for the objective function to converge to a strong minimum while maintaining a physically meaningful structure. The rotational springs did not change much during the updating process indicating a strong likelihood that their initial value was in a region of near fixity.

The updated model provided a reasonable correlation to responses monitored throughout the 2007 football season. For a full occupancy situation, it is shown that a crowd with 20% activity can produce a response above the threshold of 40 milli-g’s. The percentage is consistent with observations of actual crowd participation in the south upper deck. The model was shown to have the ability to recreate the overall frequency content and amplitude of the response. Because the loading can only be estimated roughly, it is not reasonable to expect this model to replicate a response measured in the field. What it does have the capability of doing, is providing analyst insight into the expected behavior of the structure for untested situations.
6.1: Recommendations for Clemson Memorial Stadium

Over a season’s worth of data collection of various occupancies, excitement levels, and even a few variations in forcing frequency have shown that the levels of vibrations recorded are not a concern structurally. This conclusion is reached using the updated model with a full occupancy. Examining the displacement at the tip of the cantilever during vibration levels of 40 milli-g’s, one finds a maximum displacement of 0.1 inches. For the frequency of vibration, this is well within the elastic limits of the system. The International Organization for Standards provides guidelines for sinusoidal vibration levels for structural damage and human comfort in their publication 2372 (Inman 2001). Figure 6.1 is a nomograph relating frequency, displacement, velocity, and acceleration indicating thresholds for humans and structures. 40 milli-g’s is roughly equal to 0.4 m/s². For a frequency of 2.333 Hz (the forcing frequency of the song) and an acceleration of 0.4 m/s², the resulting vibrations are just beyond the threshold for small cracks in the rendering. Rendering is a coating placed on walls that can become cracked if stresses due to vibrations pull or squeeze on the element. Since no such elements exist at the tip of the cantilever, this amount of motion is acceptable.

Concerns about decreased longevity of the structure due to increased vibrations have been raised. The findings of this research indicate that high levels of vibrations are not a new occurrence. Reactionary loadings imparted by the crowd jumping up and down in celebration were been measured with peaks reaching 37 milli-g’s and lasting 5 to 10
seconds, as seen in Figure 6.2. These types of loadings have been occurring for years (the football team was good for some of those years) without any signs of structural degradation. As long as the vibrations are monitored and levels do not exceed a recommended threshold of 40 milli-g’s, the rhythmic loading poses little threat to the structure.

Figure 6.1: Acceptable Threshold for Sinusoidal Vibrations (Inman 2001)
The experience of the fans sitting in the upper deck is of concern. Humans are sensitive to slight accelerations and can perceive accelerations as small as a few milli-g’s in a calm setting. In more lively areas, a person’s threshold for perception increases. According to the Modified Reiher–Meister Perceptibility Chart, shown in Figure 6.3, at 2.33 Hz, a sinusoidal displacement of 0.07 inches is strongly perceivable. This corresponds to an acceleration of roughly 20 milli-g’s. Depending on the tolerance level of the individual, this amount of motion will be noticeable and possible cause for concern. The reason why reactionary loadings, which have been recorded well above the threshold of perceptibility, have not generated letters of concern is due to their non-harmonic nature. It is the sustained, sinusoidal vibrations that are the most distinguishable. Taking into
consideration the lively surroundings in a stadium (which should increase the threshold of perception) and that 20 milli-g’s is not an alarming threshold; a vibration threshold of 40 milli-g’s of sustained vibrations should be set for a monitoring system.

![Modified Reiher-Meister Perceptibility Chart](image)

**Figure 6.3: Modified Reiher-Meister Perceptibility Chart (Naeim 1991)**

To monitor the vibration activity throughout the stadium, a more complete deployment of sensors is necessary. The precast system used to construct the south upper deck does not provide for a continuous system from section to section. This means that vibrations along one girder line will not be largely felt by the adjacent girder. To successfully
monitor the stadium, each girder line must be monitored independently. A single vertical
accelerometer installed at the top tip of each girder will provide a complete enough
system to monitor vibrations.

Vibrations well beyond 40 milli-g’s are not difficult to achieve. This research has
demonstrated that for a fully occupied stadium, a 20% activity level is all that is needed
to surpass the 40 milli-g threshold. Concerns about resonance can be avoided in large
part because of the high frequency of the first bending mode (3.22 Hz). A tempo of 193
beats per minute is needed to achieve resonance with this mode; a tempo difficult to
achieve and sustain. Resonance with the z-slabs can be achieved but is unlikely for two
reasons: 1) the natural frequency of the z-slabs is below 1 Hz, a tempo too slow to jump
to, and 2) the natural frequency of each z-slab is different due to increasing span length as
the girder elevation increases.

6.2: Future Work

The achievement of this work is in developing an accurate and reliable model for
evaluation of future loading scenarios. An appropriate next step is to test the robustness
of this model against various loadings. The model was tuned using the dynamic
characteristics of the stadium and should be effective at determining dynamic responses.
But how valid is the model in predicting the response of the stadium to static loads? Can
it accurately predict the response to a pluck test? By testing the model against loading it
was not tuned for, a better understanding of the true success of model updating will be seen.

This research has shown some limitations of using wind based ambient vibrations as an excitation source. A possible alternative method of excitation could be to use a small group of people in conjunction with ambient wind to excite the stadium. Research into the sensitivity of FDD to colored noise input and determining what impact, if any, the additional mass of the people have on the frequencies must be conducted to determine the validity of this.
APPENDICES
%% THIS SCRIPT ALLOWS THE USER TO INPUT ACCELERATION TIME HISTORIES OF A
%% SYSTEM IN UNITS OF G AND OUTPUT THE NAT FREQ AND MODE SHAPES.
%% ADDITIONALLY, THIS METHOD PROVIDES A MEANS OF ESTIMATING STIFFNESS
%% PARAMETERS, AND DAMPING ESTIMATES. THIS SCRIPT CALLS THE "FUNCTIONS"
%% "peakdet" "fdd" AND "zeromean"

tic
str = 'BC-2007_011.lvm';
accdata = load(str);
accData = zeromean(accdata);

%---basic data for plotting-------------
FTS = 11; % font size for plotting
scrsz = get(0,'ScreenSize');
WDT = 0.75; % width of plot
HGT = 0.30; % height of plot
TBO = 1; % start time of time history
TUP = 10; % time duration for plotting acc.

fs = 200; % sampling rate
Nfft = 4096*1; % length of fft
fc = 15; % cutting frequency

% finding natural frequency (1) : using FDD method
%--------vertical acc. CH-15 16 17 18 21 22-----------------
N_ch = 3; % no of channel
AccData = [accData(:,[4 6 9])];
disp('reading data completed. ');
% AccData = [accData(:,[3 4])];
[freq_T,SV_V,U_V] = fdd(AccData,N_ch,fs,Nfft,fc);
disp('vertical fdd done. ');

toc
%--------plotting---------------------
% h_5 = figure('Position',[10 0.1*scrsz(4) WDT*scrsz(3) HGT*scrsz(4)]);

125
%% THIS SECTION ELIMINATES THE PEAK AROUND ZERO FREQUENCY
for i=1:length(freq_T)
    if freq_T(i)<0.2
        SV_V(i)=0
    end
end

figure(h_5); plot(freq_T,SV_V);
xlabel('Frequency (Hz)', 'FontSize', FTS); ylabel('PSD', 'FontSize', FTS);
set(gca, 'FontSize', FTS);
title('SV of Horizontal Acceleration');

[row,col] = size(SV_V); % gives me the size of my PSD vector

[maxtab, mintab] = peakdet(SV_V, .0000000005);
hold on
plot((maxtab(:,1)-1)*fs/Nfft, maxtab(:,2), 'r*');
hold off

N_ch =4;  % no of channel
AccData =[accData(:,[1 2 3 4])];
% AccData = [accData(:,[3 4])];
[freq_T,SV_V,U_V]=fdd(AccData,N_ch,fs,Nfft,fc);
[r , c] = size(maxtab);

Fn = ((maxtab((1:r),1)-1).*(freq_T(1,2)-freq_T(1,1)) % possible Fn values from peak picking
phi=U_V(:,maxtab(1:r,1)) % possible mode shapes after peak picking

[ro co] =size(phi);
for i=1:co
    norm(:,i) = real(phi(:,i)) / max(abs(real(phi(:,i))));
end

norm

lam = (Fn*2*pi).^2 % rad/sec
rphi = real(phi)

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% WE WILL NOW COMPARE THE EXTRACTED MODE SHAPES WITH THE MODE SHAPES
% DETERMINED IN THE SAP MODEL UTILIZING THE MODAL ASSURANCE CRITERION AS
% ILLUSTRATED IN Caicedo 2003.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

126
raw = load('MODESHAPES.txt'); %LOAD THE TEXT FILE CONTAINING THE SAP
MODE SHAPES

N_DOF_SAP = input('input the number of DOFs included in SAP file:   ')  
N_Mode_SAP = input('input the number of modes to use from SAP file:   ')  

for i=1:N_Mode_SAP %REORDER THE SAP FILE TO A USEFUL FORMAT
    Shape(:,i)=raw(1+(i-1)*N_DOF_SAP:i*N_DOF_SAP,4);
end

DOFs = input('input the DOF numbers used to determine SVD:   ')  
Shape_SAP=Shape([DOFs,:])

for i=1:N_Mode_SAP %NORMALIZE THE MODE SHAPE
    norm_SAP(:,i) = (Shape_SAP(:,i)) / max(abs(Shape_SAP(:,i)));  
end

norm_SAP

N_plot_SAP = input('input the number of mode shapes to plot:   ')  

for i=1:N_plot_SAP  
    figure
    plot(norm_SAP(:,i))  
    hold on
    plot(norm(:,i),'r')  
    hold off
end

%NOW COMPARE THE 'norm_SAP' MODES WITH THE 'norm' MODES EXTRACTED
ABOVE.  
NORM=norm(:,1:N_Mode_SAP)  
norm_sqr = NORM.^2  
norm.sap_sqr = norm_SAP.^2  

for i = 1:N_Mode_SAP  
    for j=DOFs
        top(j,i)=NORM(j,i)*norm_SAP(j,i)
    end
end

top_sq=sum(top).^2  
bottom = sum(norm.sap_sqr).*sum(norm_sqr)  
MAC = top_sq/bottom
% THIS PORTION OF THE SCRIPT WILL IMPLEMENT THE LEAST SQUARES SOLUTION TO
% DETERMINE THE STIFFNESS PARAMETERS AS LAID OUT IN CAICEDO 2004.

\[ M = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix} \]

\[ mm = \text{input('input the number of modes to use for stiffness determination: ')} \]
\[ dd = \text{size}(\phi, 1) \]

% THIS SECTION CREATES THE THETA MATRIX FOR USE IN STIFFNESS DETERMINATION

\[
\text{for } j=1:mm \\
\quad \text{rrphi} = rphi(:, j); \\
\quad \text{for } i=1:dd \\
\quad\quad \text{the}(i,i) = \text{rrphi}(i); \\
\quad\quad \text{the}(i,i+1) = \text{rrphi}(i); \\
\quad\quad \text{if } i<\text{dd}-1 \\
\quad\quad\quad \text{ttt}(i,i+1) = \text{rrphi}(i+1); \\
\quad\quad\quad \text{end} \\
\quad\quad \text{ttt}(i+1,i+1) = \text{rrphi}(i); \\
\quad \text{end} \\
\quad \text{theta}(((j-1)*dd+1):j*dd, 1:dd) = \text{the}(1:dd, 1:dd) - \text{ttt}(1:dd, 1:dd); \\
\text{end}
\]

% THIS IS THE GAMMA FOR LOOP WORKING!!!!!!!!!!!!

\[
\text{for } i=1:mm \\
\quad \text{for } j=1:dd \\
\quad\quad \text{gamma}(j,i) = \text{rphi}(j,i) * \text{lam}(i) * \text{M}(i,i); \\
\quad \text{end} \\
\text{end} \\
\text{gam} = \text{gamma}(:); \\
\]

\[ k1 = \text{pinv}(\theta) * \text{gam} \]

% WE UTILIZE THIS NEXT SECTION TO EXTRACT THE DAMPING RATIOS. WE MUST FIRST
% DETERMINE A MORE ROBUST AUTOSPECTRA DENSITY FUNCTION MOST FIRST BE
% DETERMINED SO THAT A VALUE RICH AUTOCORRELATION FUNCTION CAN BE MAY BE
%CALCULATED USING THE IFFT. ONCE DETERMINED, LOGARITHMIC DECREMENT CAN BE
%USED TO DETERMINE THE DAMPING OF EACH MODE. THIS METHODOLOGY IS
%IMPLEMENTED BASED ON MAGALHAES et al, 2007.

%DETERMINE A MORE DATA RICH CSD. THE "2's" INDICATE THE SECOND CSD
THAT IS
%MORE DATA RICH.

fs2 = 200;  % sampling rate
Nfft2 = 4096*4;  % length of fft
fc2 = 15;  % cutting frequency

%...Setting ------------------------------------------------------
df2 = fs2 / Nfft2;
Ncut2 = floor(fc2/df2 + 1);

%...Calculate CSD Matrix ---------------------------------
for i_ch=1:N_ch
    x = AccData(:,i_ch);
y = AccData(:,i_ch);
    [Pxy2,f2]=csd(x,y,Nfft2,fs2,hanning(Nfft2),Nfft2/2);
    Spec2(i_ch,:) = Pxy2(1:Ncut2);
end
freq_T2=0:df2:f2(Ncut2);

%%%%%THIS SECTION ELIMINATES THE PEAK AROUND ZERO FREQUENCY
for i=1:length(freq_T2);
    if freq_T2(i)<.5
        Spec2(i)=0;
    end
end

for i=1:N_ch
    figure;
    plot(freq_T2,Spec2(i,:));
end

%HERE WE GO AHEAD AND BRACKET THE PEAK AND TAKE THE IFFT

for i=1:mm
    [r(i) c(i)]=find(freq_T2==Fp(i,:));
end

%PSD_WINDOW HAS THE FORM OF THE FIRST N_ch ROWS CORRESPOND TO THE FIRST
%PEAK (OR THE FIRST NATURAL FREQ). THE ROWS THAT FOLLOW ARE THE NEXT
%N_ch's OF VALUES FOR THE SECOND PEAK. THEREFORE PSD_WINDOW SHOULD BE OF
for j=1:mm
    PSD_WINDOW((j-1)*N_ch+1:(j-1)*N_ch+N_ch,:) = Spec2(:,(c(:,j)-30):(c(:,j)+30));
end

PSD=transpose(PSD_WINDOW);

zero = zeros(2^9,N_ch*mm);
for i = 1:length(PSD)
    zero(i,:) = PSD(i,:);
end

padded = transpose(zero)
for i=1:N_ch*mm

    RRxy(i,:) = real(offline(padded(i,:))); figure
    plot(RRxy(i,:))
end
function [freq,SV,U]=fdd(signal,N_ch,fs,Nfft,fc)
%...Setting -----------------------------------------------
    df  = fs / Nfft ;
    Ncut = floor(fc/df + 1) ;

%...Calculate CSD Matrix --------------------------------
for i_ch=1:N_ch
    for j_ch=i_ch:N_ch
        x = signal(:,i_ch) ;
        y = signal(:,j_ch) ;
        [Pxy,f]=csd(x,y,Nfft,fs,hanning(Nfft),Nfft/2);
        Spec(i_ch,j_ch,:) = Pxy(1:Ncut) ;
    end
end

for i_ch=2:N_ch
    for j_ch=1:(i_ch-1)
        Spec(i_ch,j_ch,:) = conj(Spec(j_ch,i_ch,:)) ;
    end
end

freq = 0:df:f(Ncut) ;
%...Calculate SV & U -------------------------------------
for ifreq = 1:length(freq)
    [u,s,v] = svd(Spec(:,:,ifreq));
    % pause
    SV(ifreq) = s(1,1);
    % pause
    U(:,ifreq) = u(:,1);
    % pause
end
% eval(['save ',infile,'.sv SV U freq df -mat']);
return;
function [maxtab, mintab]=peakdet(v, delta)

%PEAKDET Detect peaks in a vector
%        [MAXTAB, MINTAB] = PEAKDET(V, DELTA) finds the local
%        maxima and minima ("peaks") in the vector V.
%        A point is considered a maximum peak if it has the maximal
%        value, and was preceded (to the left) by a value lower by
%        DELTA. MAXTAB and MINTAB consists of two columns. Column 1
%        contains indices in V, and column 2 the found values.

% Eli Billauer, 3.4.05 (Explicitly not copyrighted).
% This function is released to the public domain; Any use is allowed.
maxtab = [];
mintab = [];
v = v(:); % Just in case this wasn't a proper vector

if (length(delta(:)))>1
    error('Input argument DELTA must be a scalar');
end

if delta <= 0
    error('Input argument DELTA must be positive');
end

mn = Inf; mx = -Inf;
mnpos = NaN; mxpos = NaN;

lookformax = 1;

for i=1:length(v)
    this = v(i);
    if this > mx, mx = this; mxpos = i; end
    if this < mn, mn = this; mnpos = i; end

    if lookformax
        if this < mx-delta
            maxtab = [maxtab ; mxpos mx];
            mn = this; mnpos = i;
            lookformax = 0;
        end
        else
            if this > mn+delta
                mintab = [mintab ; mnpos mn];
                mx = this; mxpos = i;
                lookformax = 1;
            end
        end
    end
end
clear
close all
tic
% enter the number of iterations to conduct
num_its=200;
% These are the initial estimates of the parameters in the order:
% 1) Global
% mass, 2) Point load at top 3) point load at btm 4) pnt load due to
% UDB5
% 5) E 6) A 7) I 8) spring R3 9) spring R10 10) spring R17

coeff = [1 1 1 1 1 1 1 1 1 1];
param = [1 1 1 1 4287*1.3 1 1 31278765/2 5292698/2 3698449/2];

fexp = [3.22 6.68 10.30]';

% let n be the number of parameters being modified
% let m be the number of frequencies being screened
n = 10; m = 3;

%%% Creating the weighting matrices here

% Wee is the weighting matrix for the confidence I have in each of my
% measured responses.
% Woo deals with the parameters that you are changing.

Wee(m,m) = 0;
for i=1:m
    Wee(i,i)=1/i^2;
end
% for i = 1:n
%     Wee(n,n)=0;
%     Wee(i,i)=1;
% end

Woo(n,n)=0;
Woo(1,1) = 1 ; Woo(2,2)=1 ; Woo(3,3)=1 ; Woo(4,4)=1; Woo(5,5)=.7;...
Woo(6,6)=10 ; Woo(7,7)=1 ; Woo(8,8)=1 ; Woo(9,9)=1 ; Woo(10,10)=1;

[fana] = s2k_sap_analysis(param,coeff);
coeff_new=coeff;

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% get an initial estimate of the sensitivity matrix
% I will do this by providing an initial perturbation
% to my parameters and monitor the change in frequencies.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
for i = 1:n
    disturb = .9;
    % perturb one parameter at a time by 10%
    coeff_new(i) = coeff(i)*disturb;
% enter into the S2K script with the new params and get out fana_new
    disp(strcat('Initial Disturb_',num2str(i)))
    [fana_new] = s2k_sap_analysis(param,coeff_new);
    coeff_new(i) = coeff_new(i)/disturb;
    dL_new = fana_new-fana;
    for j = 1:m
        % recognize that deltaP/P = 0.1 = disturb
        %         S(j,i) = dL_new(j)/(disturb*EI(i));
        S(j,i) = dL_new(j)/fana_new(j)/(disturb-1);
    end
end

disp(S)
dL = fexp-fana
dp = inv(S'*Wee*S+Woo)*S'*Wee*dL
coeff_new = coeff+dp'
[coeff_new]=coeffconstrain(coeff_new)
dp_act=coeff_new-coeff
for ii = 1:num_its
    disp(strcat('ITERATION_ ',num2str(ii)))
% enter into script with param_new and get out fana
    [fana] = s2k_sap_analysis(param,coeff_new);
    dL = fexp-fana;
    coeff_n = coeff_new;

    for i = 1:n
        disp(strcat('ITERATION_ ',num2str(ii),'_',num2str(i)))
% enter into script with param_n and get back fana_n
        [fana_n] = s2k_sap_analysis(param,coeff_n);
        coeff_n(i) = coeff_n(i)+ dp_act(i);

        for j = 1:m
            %                S(j,i) = dL_n(j)/dp(i);
            S(j,i) = dL_n(j)/fana_n(j)*(coeff_n(i)/dp_act(i));
        end
    end

dp = inv(S'*Wee*S+Woo)*S'*Wee*dL
coeff_new = coeff_new+dp'
[coeff_new]=coeffconstrain(coeff_new)
dp_act=coeff_new-coeff
coeff_compile(ii,:)=coeff_new;
fanacompile(ii,:)=fana;
save coeffCompilefeb16.txt coeff_compile -ASCII
save fanacompilefeb16.txt fanacompile -ASCII
for ghj=1:20
    if ii==ghj*10
send_mail_message('bsmorri', strcat('Optimization Complete', num2str(ii)), 'Matlab Done', 'fanacompilefeb16.txt')
end
end

for iii=1:n
figure
plot(coeff_compile(:,iii))
end

for i=1:length(fanacompile)
    for j=1:3
        fnresid(i,j)=((fanacompile(i,j)-fexp(j,1))/fexp(j,1))^2
    end
    fnresidsum(i)=fnresid(i,1)+fnresid(i,2)+fnresid(i,3)
end
plot(fnresidsum)
diff_freq =(fexp - fana)./fexp

send_mail_message('bsmorri', 'Optimization Complete', 'Matlab Done', 'fanacompilefeb16.txt')
toc
function [fn] = s2k_sap_analysis(initial_val,coeff)
s2k = readtextfile('GirderSub.S2K');% read the SAP s2k file into memory
(using the function 'readtextfile'

%IN THIS SECTION WE NEED TO CREATE THE NEW PARAMETERS THAT WILL BE
REPLACING THE EXISTING ONES IN THE S2K FILE
numofparams = 24; %FIGURE OUT HOW MANY LINES OF TEXT ARE GOING TO BE
CHANGED AND CREATE A MATRIX WITH THAT MANY ROWS
[row col] =size(s2k); %THIS IS USED TO FIGURE OUT HOW MANY COLUMNS WE
NEED IN THE PARAM MATRIX BECAUSE WE CAN'T SWITCH LINES OF TEXT THAT
AREN'T THE SAME LENGTH
param = char(zeros(numofparams,col)); %CREATES A MATRIX THAT IS BLANK
FOR NOW BUT OF CORRECT SIZE (COLUMN WISE) TO SWAP OUT LINES OF TEXT
WITH THE S2K FILE

for i=1:length(coeff)
  params(i)=initial_val(i)*coeff(i);
end

%MSS ALTERATIONS%
Mslab_gird=param(1); %MASS SOURCE MULTIPLIER FOR THE SLAB AND THE
GIRDER SW
Mtop=param(2); %THIS ALTERS THE POINT LOADS AT THE TOP OF THE GIRDER
DUE TO THE RAILING/WALL
Mfrntwal=param(3);
Mudb5=param(4);
Mgirdstr = strcat('   MassFrom=All   LoadCase=DEAD
Multiplier=',num2str(Mslab_gird));
Mslabstr = strcat('   MassFrom=All   LoadCase=SLABDL
Multiplier=',num2str(Mslab_gird));
Mbackwallstr = strcat('   MassFrom=All   LoadCase=BACK_WALL
Multiplier=',num2str(Mtop));
Mfrntwallstr = strcat('   MassFrom=All   LoadCase=FRONT_WALL
Multiplier=',num2str(Mfrntwal));
Mslabportstr = strcat('   MassFrom=All   LoadCase=SLAB_PORTAL_SIDE
Multiplier=',num2str(Mslab_gird));
Mudb5str = strcat('   MassFrom=All   LoadCase=UDB5
Multiplier=',num2str(Mudb5));

for i=1:length(Mgirdstr);
  param(1,i)=Mgirdstr(1,i);
end
for i=1:length(Mslabstr);
  param(2,i)=Mslabstr(1,i);
end
for i=1:length(Mbackwallstr);
  param(3,i)=Mbackwallstr(1,i);
end
for i=1:length(Mfrntwallstr);
  param(4,i)=Mfrntwallstr(1,i);

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end
for i=1:length(Mslabportstr);
    param(5,i)=Mslabportstr(1,i);
end
for i=1:length(Mudb5str);
    param(6,i)=Mudb5str(1,i);
end

%%MODULUS OF ELASTICITY ALTERATIONS%%
E = params(5);
Enew = strcat('Material=CONC   Type=Isotropic   DesignType=Concrete
UnitMass=0.0000002248   UnitWeight=0.00008681   E=
',num2str(E),'
U=0.2   A=0.0000055   MDampRatio=0   VDampMass=0   VDampStiff=0
HDampMass=0   HDampStiff=0   NumAdvance=0   Color=Green'); % USING THE EXSITING S2K TEXT AS A TEMPLATE, WE THEN INSERT A FEW VALUE IN THE APPROPRIATE PLACE
for i=1:length(Enew);
    param(7,i)=Enew(1,i);
end

%%AREA AND MOMENT OF INTERTIA MODIFICATION
A = params(6);
I = params(7);
BM1=strcat(' TotalWt=4.26698196891441   TotalMass=1.10496203964055E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM2=strcat(' TotalWt=5.63504136353106   TotalMass=1.45922969533669E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM3=strcat(' TotalWt=6.89399183600813   TotalMass=1.78524290373762E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM4=strcat(' TotalWt=3.05173716122692   TotalMass=7.90266690293528E-03
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM5=strcat(' TotalWt=3.3028746752551   TotalMass=1.74285061939838E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM6=strcat(' TotalWt=10.1871497434454   TotalMass=2.63802702721637E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
BM7=strcat(' TotalWt=9.26108115841486   TotalMass=2.39821569451867E-02
FromFile=No   AMod=',num2str(A),',   A2Mod=',num2str(A),',
A3Mod=',num2str(A),',   JMod=1   I2Mod=1   I3Mod=',num2str(I),',   MMod=1
WMod=1');
A3Mod=',num2str(A),', JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’;
BM9=strcat(’TotalWt=3.4689371491 TotalMass=0.00893032728 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
BM10=strcat(’TotalWt=3.21523157901418 TotalMass=8.32604606568814E-03 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
BM11=strcat(’TotalWt=8.61847060671614 TotalMass=2.23180761708304E-02 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
BM12=strcat(’TotalWt=6.95728047096128 TotalMass=1.80163189709952E-02 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
BM13=strcat(’TotalWt=32.598005800832 TotalMass=8.44146032027074E-02 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
BM14=strcat(’R22=1.94855715851499 ConcCol=No ConcBeam=Yes Color=Green TotalWt=0.9438766449 TotalMass=0.00244422792 FromFile=No AMod=’,num2str(A),’ A2Mod=’,num2str(A),’ A3Mod=’,num2str(A),’ JMod=1 I2Mod=1 I3Mod=’,num2str(I),’ MMod=1 WMod=1’);
for i=1:length(BM1);
    param(8,i)=BM1(1,i);
end
for i=1:length(BM2);
    param(9,i)=BM2(1,i);
end
for i=1:length(BM3);
    param(10,i)=BM3(1,i);
end
for i=1:length(BM4);
    param(11,i)=BM4(1,i);
end
for i=1:length(BM5);
    param(12,i)=BM5(1,i);
end
for i=1:length(BM6);
    param(13,i)=BM6(1,i);
end
for i=1:length(BM7);
    param(14,i)=BM7(1,i);
end
for i=1:length(BM8);
    param(15,i)=BM8(1,i);
end
for i=1:length(BM9);
    param(16,i)=BM9(1,i);
end
for i=1:length(BM10);
    param(17,i)=BM10(1,i);
end
for i=1:length(BM11);
    param(18,i)=BM11(1,i);
end
for i=1:length(BM12);
    param(19,i)=BM12(1,i);
end
for i=1:length(BM13);
    param(20,i)=BM13(1,i);
end
for i=1:length(BM14);
    param(21,i)=BM14(1,i);
end
%
\textbf{LINEAR AND ROTATIONAL SPRING BOUNDARY CONDITIONS}

\texttt{r3 = params(8);
 r10 = params(9);
 r17 = params(10);
 J3 = strcat(' Joint=3 CoordSys=Local U1=0 U2=0 U3=0 R1=0
 R2=',num2str(r3),'
 R3=0');
 J10 = strcat(' Joint=10 CoordSys=Local U1=0 U2=0 U3=0 R1=0
 R2=',num2str(r10),'
 R3=0');
 J17 = strcat(' Joint=17 CoordSys=Local U1=0 U2=0 U3=0 R1=0
 R2=',num2str(r17),'
 R3=0');
}

for i=1:length(J3);
    param(22,i)=J3(1,i);
end
for i=1:length(J10);
    param(23,i)=J10(1,i);
end
for i=1:length(J17);
    param(24,i)=J17(1,i);
end

s2k(225,:)=param(1,:);  \%WE IMPLEMENT THE PROPER ROWS OF THE PARAM
\texttt{MATRIX INTO THE S2K FILE, UPDATING THE VALUES}

s2k(226,:)=param(2,:);
s2k(227,:)=param(3,:);
s2k(228,:)=param(4,:);
s2k(229,:)=param(5,:);
s2k(230,:)=param(6,:);
s2k(69,:)=param(7,:);
s2k(148,:)=param(8,:);
s2k(150,:)=param(9,:);
s2k(152,:)=param(10,:);
s2k(154,:)=param(11,:);
s2k(156,:)=param(12,:);
s2k(158,:)=param(13,:);
s2k(160,:)=param(14,:);
s2k(162,:)=param(15,:);
s2k(164,:)=param(16,:);
s2k(166,:)=param(17,:);
s2k(168,:)=param(18,:);
s2k(170,:)=param(19,:);
s2k(172,:)=param(20,:);
s2k(178,:)=param(21,:);
s2k(345,:)=param(22,:);
s2k(346,:)=param(23,:);
s2k(347,:)=param(24,:);

%SAVE THE NEW S2K FILE OUT TO A FILE
fid = fopen('FINALGIRDER.s2k', 'w');
for i=1:row
    fprintf(fid, '%-254s\r\n', s2k(i,:));
end
fclose(fid);

display('STARTING SAP')

theprogram = '"C:\Program Files\computers and structures\sap2000
10\sap2000.exe"';
thearguments = '"C:\Stadium Vibrations\OPT WITH
SUBSTRUCTURE\FINALGIRDER.s2k/R /C /K A"';
system(sprintf('%s %s', theprogram, thearguments))

display('SAP DONE')

%AT THIS POINT, WE SHOULD HAVE ALL OF THE OUTPUT FILES FROM SAP. WE
NOW
%MUST EXTRACT THE USEFUL INFORMATION AND COMPARE IT TO THE EXPERIMENTAL
%VALUES

%NOW START PULLING IN THE RESULTS OF THE ANALYSIS

log = readtextfile('FINALGIRDER.LOG');

fn(1) = str2num(log(57,56:65));
fn(2) = str2num(log(58,56:65));
fn(3) = str2num(log(59,56:65));

fn=fn';
function [coeff]=coeffconstrain(coeff)
%GLOBAL MASS BOUNDS
if (coeff(1)>=1.5)
    coeff(1)=1.5;
end
if (coeff(1)<=0.5)
    coeff(1)=.5;
end

%POINT LOAD AT TOP BOUNDS
if (coeff(2)>=1.4)
    coeff(2)=1.4;
end
if (coeff(2)<=0.6)
    coeff(2)=.6;
end

%POINT LOAD AT BOTTOM BOUNDS
if (coeff(3)>=1.4)
    coeff(3)=1.4;
end
if (coeff(3)<=0.6)
    coeff(3)=.6;
end

%POINT LOAD DUE TO UDB5 BOUNDS
if (coeff(4)>=1.4)
    coeff(4)=1.4;
end
if (coeff(4)<=0.6)
    coeff(4)=.6;
end

%MODULUS OF ELASTICITY BOUNDS
if (coeff(5)>=1.2)
    coeff(5)=1.2;
end
if (coeff(5)<=0.7)
    coeff(5)=0.7;
end

%X-SECT AREA BOUNDS
if (coeff(6)>=1.01)
    coeff(6)=1.01;
end
if (coeff(6)<=0.99)
    coeff(6)=.99;
end

%MOMENT OF INTERTIA BOUNDS
if (coeff(7)>=1.3)
    coeff(7)=1.3;
end
if (coeff(7)<=0.7)
    coeff(7)=.7;
end

%R3 SPRING ROTATION
if (coeff(8)>=4)
coeff(8)=4;
end
if (coeff(8)<=.1)
coeff(8)=.1;
end

%R10 SPRING ROTATION
if (coeff(9)>=4)
coeff(9)=4;
end
if (coeff(9)<=.1)
coeff(9)=.1;
end

%R17 SPRING ROTATION
if (coeff(10)>=4)
coeff(10)=4;
end
if (coeff(10)<=.1)
coeff(10)=.1;
end
end
***************FUNCTION “send_mail_message”*********************

function send_mail_message(id,subject,message,attachment)
%% SEND_MAIL_MESSAGE send email to gmail once calculation is done
% Example
% send_mail_message('its.neeraj','Simulation finished','This is the message area','results.doc')

% Pradyumna
% June 2008
% Your gmail ID and password
mail = 'XXXXXX@gmail.com'; %Your GMail email address
password = 'you wish'; %Your GMail password
%From which email ID you would like to send the mail
if nargin == 1
    message = subject;
    subject = '';
elseif nargin == 2
    message = '';
    attachment = '';
elseif nargin == 3
    attachment = '';
end
% Send Mail ID
emailto = strcat(id,'@gmail.com');
% Set up Gmail SMTP service.
% Then this code will set up the preferences properly:
setpref('Internet','E_mail',mail);
setpref('Internet','SMTP_Server','smtp.gmail.com');
setpref('Internet','SMTP_Username',mail);
setpref('Internet','SMTP_Password',password);
% Gmail server.
props = java.lang.System.getProperties;
props.setProperty('mail.smtp.auth','true');
props.setProperty('mail.smtp.socketFactory.class','javax.net.ssl.SSLSocketFactory');
props.setProperty('mail.smtp.socketFactory.port','465');
% Send the email
if strcmp(mail,'GmailId@gmail.com')
    disp('Please provide your own gmail.')
    disp('You can do that by modifying the first two lines of the code')
    disp('after the comments.')
end
sendmail(emailto,subject,message,attachment)
end
APPENDIX “B”

SAP S2K FILES

***************INITIAL FINITE ELEMENT MODEL***************

File C:\Stadium Vibrations\OPT WITH SUBSTRUCTURE\GirderSub.s2k was saved on 2/16/09 at 20:48:50

TABLE: "PROGRAM CONTROL"
ProgramName=SAP2000   Version=10.0.4   ProgLevel=Advanced   LicenseOS=No
LicenseSC=No   LicenseHT=No   LicenseBR=No   CurrUnits="Kip, in, F"   SteelCode=AISC-LRFD93

TABLE: "ACTIVE DEGREES OF FREEDOM"
UX=Yes   UY=No   UZ=Yes   RX=No   RY=Yes   RZ=No

TABLE: "COORDINATE SYSTEMS"
Name=GLOBAL   Type=Cartesian   X=0   Y=0   Z=0   AboutZ=0   AboutY=0   AboutX=0

TABLE: "GRID LINES"
CoordSys=GLOBAL AxisDir=X GridID=1 XRYZCoord=0 LineType=Primary
LineColor=Gray8Dark Visible=Yes BubbleLoc=End AllVisible=Yes BubbleSize=60
CoordSys=GLOBAL AxisDir=X GridID=2 XRYZCoord=51.2496 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=3 XRYZCoord=118.17 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=4 XRYZCoord=146.184 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=5 XRYZCoord=226.1304 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=6 XRYZCoord=306.0996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=7 XRYZCoord=314.4 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=8 XRYZCoord=406.7004 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=9 XRYZCoord=507.3 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=10 XRYZCoord=529.7004 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=11 XRYZCoord=535.2996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=12 XRYZCoord=640.8 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=13 XRYZCoord=746.1996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=14 XRYZCoord=816.3996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=15 XRYZCoord=886.5996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=16 XRYZCoord=913.7004 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=17 XRYZCoord=1012.2 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=18 XRYZCoord=1109.7996 LineType=Primary
CoordSys=GLOBAL AxisDir=X GridID=19 XRYZCoord=1117.6896 LineType=Primary
TABLE: "MATERIAL PROPERTIES 01 - GENERAL"
Material=CLDFRM Type=Isotropic DesignType=ColdFormed UnitMass=7.34455290352564E-07 UnitWeight=2.83564814814815E-04 E=29500 U=0.3 A=0.0000065 MDampRatio=0 VDampMass=0 VDampStiff=0 HDampMass=0 HDampStiff=0 NumAdvance=0 Color=Red

Material=CONC Type=Isotropic DesignType=Concrete UnitMass=0.0000002248 UnitWeight=0.00008681 E=4993.1768 U=0.2 A=0.000055 MDampRatio=0 VDampMass=0 VDampStiff=0 HDampMass=0 HDampStiff=0 NumAdvance=0 Color=Green

Material=REBAR Type=Uniaxial DesignType=Rebar UnitMass=7.34455290352564E-07 UnitWeight=2.83564814814815E-04 E=29000 U=0 A=0.0000065 MDampRatio=0 VDampMass=0 VDampStiff=0 HDampMass=0 HDampStiff=0 NumAdvance=1 Color=Magenta

Material=STEEL Type=Isotropic DesignType=Steel UnitMass=7.34455290352564E-07 UnitWeight=2.83564814814815E-04 E=29000 U=0.3 A=0.0000065 MDampRatio=0 VDampMass=0 VDampStiff=0 HDampMass=0 HDampStiff=0 NumAdvance=0 Color=Blue

TABLE: "MATERIAL PROPERTIES 03 - DESIGN STEEL"
Material=STEEL Fy=36 Fu=58

TABLE: "MATERIAL PROPERTIES 04 - DESIGN CONCRETE"
Material=CONC Fc=4 RebarFy=60 RebarFys=40 LtWtConc=No LtWtFact=1

TABLE: "MATERIAL PROPERTIES 06 - DESIGN COLDFORMED"
Material=CLDFRM Fy=36 Fu=58

TABLE: "MATERIAL PROPERTIES 07 - TIME DEPENDENCE FOR STEEL"
Material=STEEL Relaxation=No Class=1

TABLE: "MATERIAL PROPERTIES 08 - TIME DEPENDENCE FOR CONCRETE"
Material=CONC E=No Creep=No Shrinkage=No S=0.25 RelHumid=50 NotionSize=3.93700787401575 BetaSC=5 ShrinkStart=0 CreepType="Full Integration"

TABLE: "MATERIAL PROPERTIES 09 - STRESS-STRAIN CURVES 1 - GENERAL"
Material=CLDFRM HysType=Kinematic Material=CONC HysType=Takeda Material=REBAR HysType=Kinematic Material=STEEL HysType=Kinematic

TABLE: "MATERIAL PROPERTIES 10 - STRESS-STRAIN CURVES 2 - DATA"
Material=CLDFRM Point=1 Strain=-0.05 Stress=0 PointID=-E Material=CLDFRM Point=2 Strain=-0.035 Stress=-18 PointID= Material=CLDFRM Point=3 Strain=-0.02 Stress=-36 PointID= Material=CLDFRM Point=4 Strain=-1.2203898305085E-03 Stress=-36 PointID=-B Material=CLDFRM Point=5 Strain=0 Stress=0 PointID=A Material=CLDFRM Point=6 Strain=0 Stress=0 PointID=B Material=CLDFRM Point=7 Strain=0 Stress=0 PointID=C Material=CLDFRM Point=8 Strain=0 Stress=0 PointID=D Material=CLDFRM Point=9 Strain=0 Stress=0 PointID=E Material=CLDFRM Point=10 Strain=0 Stress=0 PointID=F Material=CLDFRM Point=11 Strain=0 Stress=0 PointID=G Material=CLDFRM Point=12 Strain=0 Stress=0 PointID=H
Material=REBAR  Point=8  Strain=2.06896551724138E-03  Stress=60  PointID=B
Material=REBAR  Point=9  Strain=0.01  Stress=60
Material=REBAR  Point=10  Strain=0.05  Stress=90  PointID=C
Material=REBAR  Point=11  Strain=0.08  Stress=90  PointID=D
Material=REBAR  Point=12  Strain=0.1  Stress=60  PointID=E
Material=REBAR  Point=13  Strain=0.105  Stress=0  PointID=F

Material=STEEL  Point=1  Strain=-0.05  Stress=0  PointID=-E
Material=STEEL  Point=2  Strain=-0.035  Stress=-18  PointID=-D
Material=STEEL  Point=3  Strain=-0.02  Stress=58  PointID=C
Material=STEEL  Point=4  Strain=0.05  Stress=58  PointID=D
Material=STEEL  Point=5  Strain=0.1  Stress=36  PointID=E
Material=STEEL  Point=6  Strain=0.105  Stress=0  PointID=F

TABLE: "MATERIAL PROPERTIES 11 - DESIGN REBAR"
Material=REBAR  Fy=60  Fu=90

TABLE: "FRAME SECTION PROPERTIES 01 - GENERAL"
SectionName=DBLE1  Material=CONC  Shape=General  t3=39.4  t2=21  Area=827
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE15  Material=CONC  Shape=General  t3=37.82  t2=21  Area=794
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE16  Material=CONC  Shape=General  t3=46.3  t2=21  Area=972
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE18E19  Material=CONC  Shape=General  t3=50.67  t2=21  Area=1064
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE1A  Material=CONC  Shape=General  t3=47.6  t2=21  Area=999
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE20  Material=CONC  Shape=General  t3=46  t2=21  Area=966
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE21  Material=CONC  Shape=General  t3=38.8  t2=21  Area=815
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
SectionName=DBLE22  Material=CONC  Shape=General  t3=36.25  t2=21  Area=761
TorsConst=4.2669196891441 I0=1.10496203964055E-02  FromFile=No  AMod=1
A2Mod=1  A3Mod=1  JMod=1  I2Mod=1  I3Mod=0.8  MMod=1  WMod=1
### Frame Section Properties 03 - Concrete Beam

- **SectionName**: DBLE4E5
- **Material**: CONC
- **Shape**: General
- **t3**: 52.2
- **t2**: 21
- **Area**: 1096
- **TorsConst**: 1
- **I33**: 248843
- **I22**: 1
- **AS2**: 913
- **AS3**: 913
- **S33**: 1
- **S22**: 1
- **Z33**: 1
- **Z22**: 1
- **R33**: 1
- **R22**: 1
- **ConcCol**: No
- **ConcBeam**: Yes
- **Color**: Red

### Table: Link Property Definitions 01 - General

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### Table: Link Property Definitions 02 - Linear

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### Table: Load Case Definitions

- **LoadCase**: DEAD
- **DesignType**: DEAD
- **SelfWtMult**: 0
- **LoadCase**: SLABDL
- **DesignType**: DEAD
- **SelfWtMult**: 0
- **LoadCase**: BACK_WALL
- **DesignType**: DEAD
- **SelfWtMult**: 0
LoadCase=FRONT_WALL   DesignType=DEAD   SelfWtMult=0
LoadCase=SLAB_PORTAL_SIDE   DesignType=DEAD   SelfWtMult=0
LoadCase=UDBS   DesignType=DEAD   SelfWtMult=0
LoadCase=SHAKE1   DesignType=OTHER   SelfWtMult=0
LoadCase=SHAKE2   DesignType=OTHER   SelfWtMult=0

TABLE: "AUTO WAVE 3 - WAVE CHARACTERISTICS - GENERAL"
WaveChar=Default   WaveType="From Theory"   KinFactor=1   SWaterDepth=1800
WaveHeight=720   WavePeriod=12   WaveTheory=Linear

TABLE: "COMBINATION DEFINITIONS"
ComboName=SELF   ComboType="Linear Add"   CaseName="Linear Static"
CaseType="Linear Static"   CaseName=BACK_WALL   ScaleFactor=1   SteelDesign=No    ConcDesign=No   AlumDesign=No   ColdDesign=No
ComboName=SELF   CaseName=FRONT_WALL   ScaleFactor=1
ComboName=SELF   CaseName=SLABDL   ScaleFactor=1
ComboName=SELF   CaseName=DEAD   ScaleFactor=1

TABLE: "CONSTRAINT DEFINITIONS - BODY"
Name=BODY1   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=Yes   RY=Yes   RZ=Yes
Name=BODY2   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=Yes   RY=Yes   RZ=Yes
Name=BODY3   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=Yes   RY=Yes   RZ=Yes
Name=BODY4   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=No   RY=No   RZ=No
Name=BODY5   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=No   RY=No   RZ=No
Name=BODY6   CoordSys=GLOBAL   UX=Yes   UY=Yes   UZ =Yes   RX=No   RY=No   RZ=No
Name=BODY7   CoordSys=GLOBAL   UX=No   UY=No   UZ=No   RX=Yes   RY=Yes   RZ=Yes

TABLE: "MASSES 1 - MASS SOURCE"
MassFrom=All   LoadCase=DEAD   Multiplier=1.2
MassFrom=All   LoadCase=SLABDL   Multiplier=1.2
MassFrom=All   LoadCase=BACK_WALL   Multiplier=1.25
MassFrom=All   LoadCase=FRONT_WALL   Multiplier=0.8
MassFrom=All   LoadCase=SLAB_PORTAL_SIDE   Multiplier=1.2
MassFrom=All   LoadCase=UDBS   Multiplier=0.7

TABLE: "ANALYSIS CASE DEFINITIONS"
Case=DEAD   Type=LinStatic   InitialCond=Zero   RunCase=No
Case=MODAL   Type=LinModal   InitialCond=Zero   RunCase=Yes
Case=SLABDL   Type=LinStatic   InitialCond=Zero   RunCase=No
Case=FRONT_WALL   Type=LinStatic   InitialCond=Zero   RunCase=No
Case=SELF_CASE   Type=LinStatic   InitialCond=Zero   RunCase=No
Case=SLAB_PORTAL_SIDE   Type=LinStatic   InitialCond=Zero   RunCase=No
Case=UDBS   Type=LinStatic   InitialCond=Zero   RunCase=No

TABLE: "CASE - STATIC 1 - LOAD ASSIGNMENTS"
Case=DEAD   LoadType="Load case"   LoadName=DEAD   LoadSF=1
Case=SLABDL   LoadType="Load case"   LoadName=SLABDL   LoadSF=1
Case=BACK_WALL   LoadType="Load case"   LoadName=BACK_WALL   LoadSF=1
Case=FRONT_WALL   LoadType="Load case"   LoadName=FRONT_WALL   LoadSF=1
Case=SELF_CASE   LoadType="Load case"   LoadName=DEAD   LoadSF=1
Case=SELF_CASE   LoadType="Load case"   LoadName=FRONT_WALL   LoadSF=1
Case=SELF_CASE   LoadType="Load case"   LoadName=SLABDL   LoadSF=1
Case=SLAB_PORTAL_SIDE   LoadType="Load case"   LoadName=SLAB_PORTAL_SIDE   LoadSF=1
Case=UDBS   LoadType="Load case"   LoadName=UDBS   LoadSF=1

TABLE: "CASE - MODAL 1 - GENERAL"
Case=MODAL   ModeType=Eigen   MaxNumModes=12   MinNumModes=1   EigenShift=0   EigenCutoff=0   EigenTol=0.000000001   AutoShift=No

TABLE: "JOINT COORDINATES"
Joint=1   CoordSys=GLOBAL   CoordType=Cartesian   XorR=0   Y=0   Z=0   SpecialJt=Yes
GlobalX=0   GlobalY=0   GlobalZ=0
Joint=2   CoordSys=GLOBAL   CoordType=Cartesian   XorR=51.25   Y=30.1   Z=30.1   SpecialJt=Yes
GlobalX=51.25   GlobalY=0   GlobalZ=30.1
Joint=3 CoordSys=GLOBAL CoordType=Cartesian XorR=146.184 Y=0 Z=49.3 SpecialJt=Yes GlobalX=146.184 GlobalY=0 GlobalZ=49.3
Joint=4 CoordSys=GLOBAL CoordType=Cartesian XorR=118.17 Y=0 Z=69.4 SpecialJt=Yes GlobalX=118.17 GlobalY=0 GlobalZ=69.4
Joint=5 CoordSys=GLOBAL CoordType=Cartesian XorR=132.177 Y=0 Z=78.85 SpecialJt=No GlobalX=132.177 GlobalY=0 GlobalZ=78.85
Joint=6 CoordSys=GLOBAL CoordType=Cartesian XorR=146.184 Y=0 Z=88.3 SpecialJt=Yes GlobalX=146.184 GlobalY=0 GlobalZ=88.3
Joint=7 CoordSys=GLOBAL CoordType=Cartesian XorR=226.13 Y=0 Z=147.5 SpecialJt=Yes GlobalX=226.13 GlobalY=0 GlobalZ=147.5
Joint=8 CoordSys=GLOBAL CoordType=Cartesian XorR=146.184 Y=0 Z=-655.32 SpecialJt=No GlobalX=146.184 GlobalY=0 GlobalZ=-655.32
Joint=9 CoordSys=GLOBAL CoordType=Cartesian XorR=406.7 Y=0 Z=274.4 SpecialJt=Yes GlobalX=406.7 GlobalY=0 GlobalZ=274.4
Joint=10 CoordSys=GLOBAL CoordType=Cartesian XorR=529.7 Y=0 Z=329.5 SpecialJt=Yes GlobalX=529.7 GlobalY=0 GlobalZ=329.5
Joint=11 CoordSys=GLOBAL CoordType=Cartesian XorR=507.3 Y=0 Z=342 SpecialJt=Yes GlobalX=507.3 GlobalY=0 GlobalZ=342
Joint=12 CoordSys=GLOBAL CoordType=Cartesian XorR=521.3 Y=0 Z=351.4 SpecialJt=No GlobalX=521.3 GlobalY=0 GlobalZ=351.4
Joint=13 CoordSys=GLOBAL CoordType=Cartesian XorR=535.3 Y=0 Z=360.8 SpecialJt=Yes GlobalX=535.3 GlobalY=0 GlobalZ=360.8
Joint=14 CoordSys=GLOBAL CoordType=Cartesian XorR=640.8 Y=0 Z=431.8 SpecialJt=Yes GlobalX=640.8 GlobalY=0 GlobalZ=431.8
Joint=15 CoordSys=GLOBAL CoordType=Cartesian XorR=946.2 Y=0 Z=502.8 SpecialJt=Yes GlobalX=946.2 GlobalY=0 GlobalZ=502.8
Joint=16 CoordSys=GLOBAL CoordType=Cartesian XorR=816.4 Y=0 Z=544.7 SpecialJt=Yes GlobalX=816.4 GlobalY=0 GlobalZ=544.7
Joint=17 CoordSys=GLOBAL CoordType=Cartesian XorR=913.7 Y=0 Z=567 SpecialJt=Yes GlobalX=913.7 GlobalY=0 GlobalZ=567
Joint=18 CoordSys=GLOBAL CoordType=Cartesian XorR=886.6 Y=0 Z=586.5 SpecialJt=Yes GlobalX=886.6 GlobalY=0 GlobalZ=586.5
Joint=19 CoordSys=GLOBAL CoordType=Cartesian XorR=900.15 Y=0 Z=595.95 SpecialJt=No GlobalX=900.15 GlobalY=0 GlobalZ=595.95
Joint=20 CoordSys=GLOBAL CoordType=Cartesian XorR=913.7 Y=0 Z=605.4 SpecialJt=Yes GlobalX=913.7 GlobalY=0 GlobalZ=605.4
Joint=21 CoordSys=GLOBAL CoordType=Cartesian XorR=1012.2 Y=0 Z=676.5 SpecialJt=Yes GlobalX=1012.2 GlobalY=0 GlobalZ=676.5
Joint=22 CoordSys=GLOBAL CoordType=Cartesian XorR=1117.69 Y=0 Z=754 SpecialJt=Yes GlobalX=1117.69 GlobalY=0 GlobalZ=754
Joint=23 CoordSys=GLOBAL CoordType=Cartesian XorR=1170.2 Y=0 Z=754 SpecialJt=Yes GlobalX=1170.2 GlobalY=0 GlobalZ=754
Joint=24 CoordSys=GLOBAL CoordType=Cartesian XorR=-75.5 Y=0 Z=0 SpecialJt=Yes GlobalX=-75.5 GlobalY=0 GlobalZ=0
Joint=25 CoordSys=GLOBAL CoordType=Cartesian XorR=1170.2 Y=0 Z=748.125 SpecialJt=Yes GlobalX=1170.2 GlobalY=0 GlobalZ=748.125
Joint=26 CoordSys=GLOBAL CoordType=Cartesian XorR=1170.2 Y=0 Z=848.375 SpecialJt=Yes GlobalX=1170.2 GlobalY=0 GlobalZ=848.375
Joint=27 CoordSys=GLOBAL CoordType=Cartesian XorR=-75.5 Y=0 Z=747.7 SpecialJt=No GlobalX=-75.5 GlobalY=0 GlobalZ=747.7
Joint=28 CoordSys=GLOBAL CoordType=Cartesian XorR=529.7004 Y=0 Z=-655.32 SpecialJt=No GlobalX=529.7004 GlobalY=0 GlobalZ=-655.32
Joint=29 CoordSys=GLOBAL CoordType=Cartesian XorR=913.7004 Y=0 Z=-655.32 SpecialJt=No GlobalX=913.7004 GlobalY=0 GlobalZ=-655.32
Joint=30 CoordSys=GLOBAL CoordType=Cartesian XorR=-75.5 Y=0 Z=-655.32 SpecialJt=Yes GlobalX=-75.5 GlobalY=0 GlobalZ=-655.32
Joint=100 CoordSys=GLOBAL CoordType=Cartesian XorR=306.1 Y=0 Z=206.8 SpecialJt=Yes GlobalX=306.1 GlobalY=0 GlobalZ=206.8

TABLE: "CONNECTIVITY - FRAME"
Frame=1 JointI=26 JointJ=23 IsCurved=No Length=94.375 CentroidX=1170.2 CentroidY=801.1875
Frame=2 JointI=23 JointJ=27 IsCurved=No Length=6.29999999999995 CentroidX=1170.2 CentroidY=750.85
Frame=6 JointI=30 JointJ=8 IsCurved=No Length=221.684 CentroidX=35.342 CentroidY=0
Frame=7   JointI=8   JointJ=28   IsCurved=No   Length=383.5164   CentroidX=337.9422
Frame=8   JointI=28   JointJ=29   IsCurved=No   Length=384   CentroidX=721.7004
Frame=E1   JointI=1   JointJ=2   IsCurved=No   Length=59.4354481769928   CentroidX=25.625   CentroidY=0   CentroidZ=15.05
Frame=E4   JointI=4   JointJ=5   IsCurved=No   Length=77.6065486927489   CentroidX=125.1735   CentroidY=0   CentroidZ=83.575
Frame=E5   JointI=5   JointJ=6   IsCurved=No   Length=16.8967023113979   CentroidX=139.1805   CentroidY=0   CentroidZ=83.575
Frame=E6   JointI=6   JointJ=7   IsCurved=No   Length=99.4786555799786   CentroidX=186.157   CentroidY=0   CentroidZ=117.9
Frame=E7   JointI=7   JointJ=100   IsCurved=No   Length=99.5574753597137   CentroidX=266.115   CentroidY=0   CentroidZ=177.15
Frame=E8   JointI=100   JointJ=9   IsCurved=No   Length=121.20805248063   CentroidX=356.4   CentroidY=0   CentroidZ=240.6
Frame=E9   JointI=9   JointJ=11   IsCurved=No   Length=121.20805248063   CentroidX=457   CentroidY=0   CentroidZ=308.2
Frame=E11   JointI=11   JointJ=12   IsCurved=No   Length=16.862977198585   CentroidX=514.3   CentroidY=0   CentroidZ=346.7
Frame=E12   JointI=12   JointJ=13   IsCurved=No   Length=16.862977198585   CentroidX=528.3   CentroidY=0   CentroidZ=356.1
Frame=E13   JointI=13   JointJ=14   IsCurved=No   Length=127.16622973932   CentroidX=588.05   CentroidY=0   CentroidZ=396.3
Frame=E14   JointI=14   JointJ=15   IsCurved=No   Length=127.08327978141   CentroidX=693.5   CentroidY=0   CentroidZ=467.3
Frame=E15   JointI=15   JointJ=16   IsCurved=No   Length=81.753593192046   CentroidX=781.3   CentroidY=0   CentroidZ=523.75
Frame=E16   JointI=16   JointJ=18   IsCurved=No   Length=81.7023867460431   CentroidX=851.5   CentroidY=0   CentroidZ=565.6
Frame=E18   JointI=18   JointJ=19   IsCurved=No   Length=16.5198365609348   CentroidX=893.375   CentroidY=0   CentroidZ=591.225
Frame=E19   JointI=19   JointJ=20   IsCurved=No   Length=16.5198365609349   CentroidX=906.925   CentroidY=0   CentroidZ=600.675
Frame=E1A   JointI=24   JointJ=1   IsCurved=No   Length=75.5   CentroidX=1143.945   CentroidY=0   CentroidZ=754
Frame=E20   JointI=20   JointJ=21   IsCurved=No   Length=121.48028646657   CentroidX=1064.945   CentroidY=0   CentroidZ=715.25
Frame=E21   JointI=21   JointJ=22   IsCurved=No   Length=130.8983960907   CentroidX=1064.945   CentroidY=0   CentroidZ=715.25
Frame=E22   JointI=22   JointJ=23   IsCurved=No   Length=52.51   CentroidX=1143.945   CentroidY=0   CentroidZ=754

TABLE: "JOINT CONSTRAINT ASSIGNMENTS"
Joint=3   Constraint=BODY1   Type=Body
Joint=3   Constraint=BODY4   Type=Body
Joint=5   Constraint=BODY1   Type=Body
Joint=8   Constraint=BODY4   Type=Body
Joint=10   Constraint=BODY2   Type=Body
Joint=10   Constraint=BODY5   Type=Body
Joint=12   Constraint=BODY2   Type=Body
Joint=17   Constraint=BODY3   Type=Body
Joint=17   Constraint=BODY6   Type=Body
Joint=19   Constraint=BODY3   Type=Body
Joint=19   Constraint=BODY7   Type=Body
Joint=23   Constraint=BODY3   Type=Body
Joint=26   Constraint=BODY7   Type=Body
Joint=28   Constraint=BODY4   Type=Body
Joint=28   Constraint=BODY5   Type=Body
Joint=29   Constraint=BODY4   Type=Body
Joint=29   Constraint=BODY6   Type=Body

TABLE: "JOINT RESTRAINT ASSIGNMENTS"
Joint=3   U1=Yes   U2=Yes   U3=Yes   R1=No   R2=No   R3=No
Joint=8   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
Joint=10   U1=No   U2=No   U3=Yes   R1=No   R2=No   R3=No
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**TABLE: "JOINT SPRING ASSIGNMENTS 1 - UNCOUPLED"**

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**TABLE: "JOINT LOADS - FORCE"**

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MATProp | Default
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**TABLE: "FRAME AUTO MESH ASSIGNMENTS"**

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### Table: "Frame Loads - Distributed"

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</tr>
<tr>
<td>E9</td>
<td>SLAB_PORTAL_SIDE</td>
<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>121.202805248063</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>FOverLA = 8.376666666666667E-02, FOverLB = 0.08485</td>
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</table>

}````
Frame=E11   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=16.862977198585
FOverLA=0.08485   FOverLB=0.085
Frame=E12   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=16.8629771985851
FOverLA=0.085   FOverLB=8.51583333333333E-02
Frame=E12   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=16.8629771985851
FOverLA=0.085   FOverLB=8.51583333333333E-02
Frame=E13   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=127.16622979392
FOverLA=8.51583333333333E-02   FOverLB=8.62916666666667E-02
Frame=E13   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=127.16622979392
FOverLA=8.51583333333333E-02   FOverLB=8.62916666666667E-02
Frame=E14   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=127.08327978141
FOverLA=8.62916666666667E-02   FOverLB=8.74333333333333E-02
Frame=E14   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=127.08327978141
FOverLA=8.62916666666667E-02   FOverLB=8.74333333333333E-02
Frame=E15   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=81.7535931932046
FOverLA=8.74333333333333E-02   FOverLB=8.81666666666667E-02
Frame=E15   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=81.7535931932046
FOverLA=8.74333333333333E-02   FOverLB=8.81666666666667E-02
Frame=E16   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=81.70238365609348
FOverLA=8.81666666666667E-02   FOverLB=0.0889
Frame=E16   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=81.70238365609348
FOverLA=8.81666666666667E-02   FOverLB=0.0889
Frame=E18   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=16.5198365609349
FOverLA=8.90416666666667E-02   FOverLB=8.91916666666667E-02
Frame=E18   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=16.5198365609349
FOverLA=8.90416666666667E-02   FOverLB=8.91916666666667E-02
Frame=E19   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=52.51099999999998
FOverLA=6.51666666666667E-02   FOverLB=6.51666666666667E-02
Frame=E19   LoadCase=SLAB_PORTAL_SIDE   CoordSys=GLOBAL   Type=Force   Dir=Gravity
DistType=RelDist   RelDistA=0   RelDistB=1   AbsDistA=0   AbsDistB=52.51099999999998
FOverLA=6.51666666666667E-02   FOverLB=6.51666666666667E-02

TABLE:  "FRAME DESIGN PROCEDURES"

Frame=1   DesignProc="From Material"
Frame=2   DesignProc="From Material"
TABLE: "OVERWRITES - STEEL DESIGN - AISC-LRFD93"

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<th>DesignSect</th>
<th>FrameType</th>
<th>RLLF</th>
<th>AreaRatio</th>
<th>XLMajor</th>
<th>XLMinor</th>
<th>XKMajor</th>
<th>XKMinor</th>
<th>CmMajor</th>
<th>CmMinor</th>
<th>Cb</th>
<th>B1Major</th>
<th>B1Minor</th>
<th>B2Major</th>
<th>B2Minor</th>
<th>PhiPnc</th>
<th>PhiPnt</th>
<th>PhiMn3</th>
<th>PhiMn2</th>
<th>PhiVn2</th>
<th>PhiVn3</th>
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<tbody>
<tr>
<td>Frame 6</td>
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<td>FrameType=&quot;Program Determined&quot;</td>
<td>Fy=0</td>
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<td>Cb=0</td>
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<td>B1Minor=0</td>
<td>B2Major=0</td>
<td>B2Minor=0</td>
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<td>PhiPnt=0</td>
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<td>PhiMn2=0</td>
<td>PhiVn2=0</td>
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<td>B1Minor=0</td>
<td>B2Major=0</td>
<td>B2Minor=0</td>
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<td>PhiPnt=0</td>
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<td>PhiMn2=0</td>
<td>PhiVn2=0</td>
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TABLE: "OVERWRITES - CONCRETE DESIGN - ACI 318-05/IBC 2003"

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<td>Frame 3</td>
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<td>FrameType=&quot;Program Determined&quot;</td>
<td>RLLF=0</td>
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END TABLE DATA
FILE C:\Stadium Vibrations\OPT WITH SUBSTRUCTURE\GirderSub.s2k was saved on 2/16/09 at 20:48:50

**TABLE: "PROGRAM CONTROL"**
ProgramName=SAP2000  Version=10.0.4  ProgLevel=Advanced  LicenseOS=No

**TABLE: "ACTIVE DEGREES OF FREEDOM"**
UX=Yes  UY=No  UZ=Yes  RX=No  RY=Yes  RZ=No

**TABLE: "COORDINATE SYSTEMS"**
Name=GLOBAL  Type=Cartesian  X=0  Y=0  Z=0  AboutZ=0  AboutY=0  AboutX=0

**TABLE: "GRID LINES"**
CoordSys=GLOBAL  AxisDir=X  GridID=1  XRYZCoord=0  LineType=Primary  LineColor=Gray8Dark  Visible=Yes  BubbleLoc=End  AllVisible=Yes  BubbleSize=60
CoordSys=GLOBAL  AxisDir=X  GridID=2  XRYZCoord=51.2496  LineType=Primary  LineColor=Gray8Dark  Visible=Yes  BubbleLoc=End
CoordSys=GLOBAL  AxisDir=X  GridID=3  XRYZCoord=118.17  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=4  XRYZCoord=146.184  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=5  XRYZCoord=226.1304  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=6  XRYZCoord=306.0996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=7  XRYZCoord=314.4  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=8  XRYZCoord=406.7004  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=9  XRYZCoord=507.3  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=10  XRYZCoord=529.7004  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=11  XRYZCoord=535.2996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=12  XRYZCoord=640.8  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=13  XRYZCoord=746.1996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=14  XRYZCoord=816.3996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=15  XRYZCoord=886.5996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=16  XRYZCoord=913.7004  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=17  XRYZCoord=1012.2  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=18  XRYZCoord=1109.7996  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=19  XRYZCoord=1117.6896  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=20  XRYZCoord=1117.6896  LineType=Primary
CoordSys=GLOBAL  AxisDir=X  GridID=21  XRYZCoord=1170.2004  LineType=Primary
CoordSys=GLOBAL  AxisDir=Y  XRYZCoord=0  LineType=Primary  LineColor=Gray8Dark  Visible=Yes  BubbleLoc=Start
CoordSys=GLOBAL  AxisDir=Z  XRYZCoord=655.32  LineType=Secondary
CoordSys=GLOBAL  AxisDir=Z  XRYZCoord=309.7599996  LineType=Secondary
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### TABLE: "MATERIAL PROPERTIES 03 - DESIGN STEEL"

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<th>Type</th>
<th>DesignType</th>
<th>UnitMass</th>
<th>UnitWeight</th>
<th>E</th>
<th>U</th>
<th>A</th>
<th>MDampRatio</th>
<th>Color</th>
</tr>
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<tbody>
<tr>
<td>REBAR</td>
<td>Uniaxial</td>
<td>Rebar</td>
<td>7.3455290352564E-07</td>
<td>2.83564814814815E-04</td>
<td>29000</td>
<td>0.3</td>
<td>0.0000065</td>
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<td>Magenta</td>
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</table>

### TABLE: "MATERIAL PROPERTIES 04 - DESIGN CONCRETE"

<table>
<thead>
<tr>
<th>Material</th>
<th>Fy</th>
<th>Fu</th>
</tr>
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<tbody>
<tr>
<td>CONC</td>
<td>36</td>
<td>58</td>
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### TABLE: "MATERIAL PROPERTIES 06 - DESIGN COLDFORMED"

<table>
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<th>DesignType</th>
<th>UnitMass</th>
<th>UnitWeight</th>
<th>E</th>
<th>U</th>
<th>A</th>
<th>MDampRatio</th>
<th>Color</th>
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</thead>
<tbody>
<tr>
<td>CLDFRM</td>
<td>Kinematic</td>
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<td></td>
<td></td>
<td>29000</td>
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<td>0.0000065</td>
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### TABLE: "MATERIAL PROPERTIES 07 - TIME DEPENDENCE FOR STEEL"

<table>
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<tr>
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### TABLE: "MATERIAL PROPERTIES 08 - TIME DEPENDENCE FOR CONCRETE"

<table>
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<th>Creep</th>
<th>Shrinkage</th>
<th>S</th>
<th>RelHumid</th>
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<th>CreepType</th>
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<tr>
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<td>No</td>
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### TABLE: "MATERIAL PROPERTIES 09 - STRESS-STRAIN CURVES 1 - GENERAL"

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<td>CONC</td>
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### TABLE: "MATERIAL PROPERTIES 10 - STRESS-STRAIN CURVES 2 - DATA"

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<td>CLDFRM</td>
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<td>A</td>
</tr>
<tr>
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Material=STEEL  Point=5  Strain=0  Stress=0  PointID=H
Material=STEEL  Point=6  Strain=0.02  Stress=36  PointID=B
Material=STEEL  Point=7  Strain=1.24137931034483E-03  Stress=36  PointID=D
Material=STEEL  Point=8  Strain=0.08  Stress=58  PointID=C
Material=STEEL  Point=9  Strain=0.1  Stress=36  PointID=E
Material=STEEL  Point=10  Strain=0.105  Stress=0  PointID=F
Material=STEEL  Point=11  Strain=0.105  Stress=0  PointID=G

TABLE:  "MATERIAL PROPERTIES 11 - DESIGN REBAR"
Material=REBAR   Fy=60   Fu=90

TABLE:  "FRAME SECTION PROPERTIES 01 - GENERAL"
SectionName=DBLE1  Material=CONC   Shape=General  t3=39.4  t2=21  Area=827  TorsConst=1  I33=109545  I22=1  AS2=689  AS3=689  S33=1  S22=1  Z33=1  Z22=1  R33=1  R22=1  ConcCol=No  ConcBeam=No  Color=Red  TotalWt=4.266981968914411  TotalMass=1.10496203964055E-02  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1
SectionName=DBLE15  Material=CONC   Shape=General  t3=46.3  t2=21  Area=972  TorsConst=1  I33=97992  I22=1  AS2=662  AS3=662  S33=1  S22=1  Z33=1  Z22=1  R33=1  R22=1  ConcCol=No  ConcBeam=No  Color=Red  TotalWt=5.63504113653106  TotalMass=1.4592269533669E-02  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1
SectionName=DBLE16  Material=CONC   Shape=General  t3=50.67  t2=21  Area=1064  TorsConst=1  I33=815  I22=1  AS2=679  AS3=679  S33=1  S22=1  Z33=1  Z22=1  R33=1  R22=1  ConcCol=No  ConcBeam=Yes  Color=Green  TotalWt=9.261081158411491  TotalMass=2.326065014584186E-03  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1
SectionName=DBLE18E19  Material=CONC   Shape=Rectangular  t3=24  t2=21  Area=504  TorsConst=35242.0108538818  I33=24192  I22=18522  AS2=420  AS3=420  S33=2016  S22=1764  Z33=3024  Z22=2646  R33=6.92820323027551  R22=6.06217782649107  ConcCol=No  ConcBeam=Yes  Color=Green  TotalWt=3.305173716122952  TotalMass=0.0085540896  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1
SectionName=DBLE2  Material=CONC   Shape=General  t3=47.6  t2=21  Area=999  TorsConst=1  I33=193758  I22=1  AS2=832  AS3=832  S33=1  S22=1  Z33=1  Z22=1  R33=1  R22=1  ConcCol=No  ConcBeam=No  Color=Red  TotalWt=6.89399183600813  TotalMass=1.7852490373762E-02  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1
SectionName=DBLE20  Material=CONC   Shape=General  t3=38.8  t2=21  Area=815  TorsConst=1  I33=103454  I22=1  AS2=679  AS3=679  S33=1  S22=1  Z33=1  Z22=1  R33=1  R22=1  ConcCol=No  ConcBeam=No  Color=Red  TotalWt=9.261081158411491  TotalMass=2.326065014584186E-03  FromFile=No  AMod=0.99231  A2Mod=0.99231  A3Mod=0.99231  JMod=1  I2Mod=1  I3Mod=0.81519  MMod=1  WMod=1

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SectionName=DBLE6   Material=CONC   Shape=General   t3=47.55   t2=21   Area=998
TorsConst=1   I33=47351   I22=1   AS2=2317   AS3=2317   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Red   TotalWt=8.61847060671614   TotalMass=2.23180761708304E-02   FromFile=No   AMod=0.99231
A2Mod=0.99231   A3Mod=0.99231   JMod=1   I2Mod=1   I3Mod=0.81519   MMod=1   WMod=1
SectionName=DBLE7   Material=CONC   Shape=General   t3=38.32   t2=21   Area=805
TorsConst=1   I33=102718   I22=1   AS2=671   AS3=671   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Red   TotalWt=6.95728047096128   TotalMass=1.80163189709952E-02   FromFile=No   AMod=0.99231
A2Mod=0.99231   A3Mod=0.99231   JMod=1   I2Mod=1   I3Mod=0.81519   MMod=1   WMod=1
SectionName=DBLE8E14   Material=CONC   Shape=General   t3=33.7   t2=21   Area=708
TorsConst=1   I33=66977   I22=1   AS2=590   AS3=590   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Red   TotalWt=3.2598005800832   TotalMass=8.44146032027074E-02   FromFile=No   AMod=0.99231
A2Mod=0.99231   A3Mod=0.99231   JMod=1   I2Mod=1   I3Mod=0.81519   MMod=1   WMod=1
SectionName=LOWSPRING   Material=STEEL   Shape=General   t3=1   t2=1   Area=2.81095
TorsConst=1   I33=1   I22=1   AS2=1   AS3=1   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Green   TotalWt=0   TotalMass=0   FromFile=No   AMod=1   A2Mod=1   A3Mod=1   JMod=1
I2Mod=1   I3Mod=1   MMod=1   WMod=1
SectionName=MIDSPRING   Material=STEEL   Shape=General   t3=1   t2=1   Area=1.5618
TorsConst=1   I33=1   I22=1   AS2=1   AS3=1   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Green   TotalWt=0   TotalMass=0   FromFile=No   AMod=1   A2Mod=1   A3Mod=1   JMod=1
I2Mod=1   I3Mod=1   MMod=1   WMod=1
SectionName=TOPPOST   Material=CONC   Shape=Rectangular   t3=16   t2=6.75   Area=108
TorsConst=1205.4530740759   I33=2304   I22=410.0625   AS2=90   AS3=90   S33=288   S22=121.5   Z33=432   Z22=182.25   R33=4.618880215351701
ConcCol=No   ConcBeam=Yes   Color=Green   TotalWt=0.943876449   TotalMass=0.00244422792   FromFile=No   AMod=0.99231
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SectionName=TOPSPRING   Material=STEEL   Shape=General   t3=1   t2=1   Area=0.59453
TorsConst=1   I33=1   I22=1   AS2=1   AS3=1   S33=1   S22=1   Z33=1   Z22=1   R33=1   R22=1
ConcCol=No   ConcBeam=No   Color=Green   TotalWt=0   TotalMass=0   FromFile=No   AMod=1   A2Mod=1   A3Mod=1   JMod=1
I2Mod=1   I3Mod=1   MMod=1   WMod=1
TABLE:  "FRAME SECTION PROPERTIES 03 - CONCRETE BEAM"
SectionName=DBLE1A   RebarMat=REBAR   TopCover=2.4   BotCover=2.4   TopLeftArea=0   TopRightArea=0   BotLeftArea=0   BotRightArea=0
SectionName=TOPPOST   RebarMat=REBAR   TopCover=19.2   BotCover=19.2   TopLeftArea=0   TopRightArea=0   BotLeftArea=0   BotRightArea=0
TABLE:  "LINK PROPERTY DEFINITIONS 01 - GENERAL"
Link=lowspring   LinkType=Linear   Mass=0   Weight=0   RotInert1=0   RotInert2=0
RotInert3=0   PDM2I=0   PDM2J=0   PDM3I=0   PDM3J=0   Color=Magenta
Link=midspring   LinkType=Linear   Mass=0   Weight=0   RotInert1=0   RotInert2=0
RotInert3=0   PDM2I=0   PDM2J=0   PDM3I=0   PDM3J=0   Color=Yellow
Link=topspring   LinkType=Linear   Mass=0   Weight=0   RotInert1=0   RotInert2=0
RotInert3=0   PDM2I=0   PDM2J=0   PDM3I=0   PDM3J=0   Color=White
TABLE:  "LINK PROPERTY DEFINITIONS 02 - LINEAR"
Link=lowspring   DOF=U1   Fixed=No   TransKE=726   TransCE=0
Link=midspring   DOF=U1   Fixed=No   TransKE=165   TransCE=0
Link=topspring   DOF=U1   Fixed=No   TransKE=214   TransCE=0
TABLE:  "LOAD CASE DEFINITIONS"
LoadCase=DEAD   DesignType=DEAD   SelfWtMult=0
LoadCase=SLABDL   DesignType=DEAD   SelfWtMult=0
LoadCase=FRONT_WALL   DesignType=DEAD   SelfWtMult=0
LoadCase=SLAB сторона_стена   DesignType=DEAD   SelfWtMult=0
LoadCase=UDS   DesignType=DEAD   SelfWtMult=0
LoadCase=SHARE1   DesignType=OTHER   SelfWtMult=0
LoadCase=SHARE2   DesignType=OTHER   SelfWtMult=0
TABLE:  "AUTO WAVE 3 - WAVE CHARACTERISTICS - GENERAL"
**TABLE: "COMBINATION DEFINITIONS"**

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**TABLE: "MASSES 1 - MASS SOURCE"**

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**TABLE: "ANALYSIS CASE DEFINITIONS"**

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**TABLE: "CASE - STATIC 1 - LOAD ASSIGNMENTS"**

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**TABLE: "CASE - MODAL 1 - GENERAL"**

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Joint=9  CoordSys=GLOBAL CoordType=Cartesian XorR=406.7  Y=0  Z=274.4
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Joint=11  CoordSys=GLOBAL CoordType=Cartesian XorR=507.3  Y=0  Z=342
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Joint=12  CoordSys=GLOBAL CoordType=Cartesian XorR=521.3  Y=0  Z=351.4
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Joint=13  CoordSys=GLOBAL CoordType=Cartesian XorR=535.3  Y=0  Z=360.8
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Joint=14  CoordSys=GLOBAL CoordType=Cartesian XorR=640.8  Y=0  Z=431.8
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Joint=15  CoordSys=GLOBAL CoordType=Cartesian XorR=746.2  Y=0  Z=502.8
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Joint=17  CoordSys=GLOBAL CoordType=Cartesian XorR=913.7  Y=0  Z=567
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  SpecialJt=Yes  GlobalX=1170.2  GlobalY=0  GlobalZ=748.125
Joint=26  CoordSys=GLOBAL CoordType=Cartesian XorR=1170.2  Y=0  Z=848.375
  SpecialJt=Yes  GlobalX=1170.2  GlobalY=0  GlobalZ=848.375
Joint=27  CoordSys=GLOBAL CoordType=Cartesian XorR=1170.2  Y=0  Z=747.7
  SpecialJt=No  GlobalX=1170.2  GlobalY=0  GlobalZ=747.7
Joint=28  CoordSys=GLOBAL CoordType=Cartesian XorR=529.7004  Y=0  Z=-655.32
  SpecialJt=No  GlobalX=529.7004  GlobalY=0  GlobalZ=-655.32
Joint=29  CoordSys=GLOBAL CoordType=Cartesian XorR=529.7004  Y=0  Z=-655.32
  SpecialJt=No  GlobalX=529.7004  GlobalY=0  GlobalZ=-655.32
Joint=30  CoordSys=GLOBAL CoordType=Cartesian XorR=529.7004  Y=0  Z=-655.32
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Joint=100 CoordSys=GLOBAL CoordType=Cartesian XorR=306.1  Y=0  Z=206.8
  SpecialJt=Yes  GlobalX=306.1  GlobalY=0  GlobalZ=206.8

TABLE: "CONNECTIVITY - FRAME"
Frame=1  JointI=26  JointJ=23  IsCurved=No  Length=94.375  CentroidX=1170.2
  CentroidY=0  CentroidZ=748.125
Frame=2  JointI=23  JointJ=27  IsCurved=No  Length=6.29999999999999
  CentroidX=1170.2  CentroidY=0  CentroidZ=748.125
Frame=6  JointI=30  JointJ=8  IsCurved=No  Length=221.684  CentroidX=35.342
  CentroidY=0  CentroidZ=35.342
Frame=7  JointI=8  JointJ=28  IsCurved=No  Length=383.5164  CentroidX=337.9422
  CentroidY=0  CentroidZ=337.9422
Frame=8  JointI=28  JointJ=29  IsCurved=No  Length=384  CentroidX=721.7004
  CentroidY=0  CentroidZ=721.7004
Frame=E1  JointI=1  JointJ=2  IsCurved=No  Length=59.4354481769928
  CentroidX=25.625  CentroidY=0  CentroidZ=15.05
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  CentroidX=84.71  CentroidY=0  CentroidZ=49.75
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Centroidx=125.1735 Centroidy=0 Centroidz=74.125
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Centroidx=139.1805 Centroidy=0 Centroidz=83.575
Frame=E6 JointI=6 JointJ=7 IsCurved=No Length=99.4786555799786
Centroidx=266.115 Centroidy=0 Centroidz=177.15
Frame=E7 JointI=7 JointJ=100 IsCurved=No Length=99.5574753597137
Centroidx=356.4 Centroidy=0 Centroidz=240.6
Frame=E8 JointI=9 JointJ=11 IsCurved=No Length=121.202805248063
Centroidx=457 Centroidy=0 Centroidz=308.2
Frame=E9 JointI=10 JointJ=11 IsCurved=No Length=121.202805248063
Centroidx=528.3 Centroidy=0 Centroidz=356.1
Frame=E10 JointI=11 JointJ=12 IsCurved=No Length=16.862977198585
Centroidx=514.3 Centroidy=0 Centroidz=346.7
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Centroidx=528.3 Centroidy=0 Centroidz=356.1
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Centroidx=588.05 Centroidy=0 Centroidz=396.3
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Centroidx=693.5 Centroidy=0 Centroidz=467.3
Frame=E15 JointI=15 JointJ=16 IsCurved=No Length=81.7535931932046
Centroidx=781.3 Centroidy=0 Centroidz=523.75
Frame=E16 JointI=16 JointJ=18 IsCurved=No Length=81.7023867460431
Centroidx=851.5 Centroidy=0 Centroidz=565.6
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Centroidx=906.925 Centroidy=0 Centroidz=600.675
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Centroidy=0 Centroidz=0
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Centroidx=962.95 Centroidy=0 Centroidz=640.95
Frame=E21 JointI=21 JointJ=22 IsCurved=No Length=130.89839609407
Centroidx=1064.945 Centroidy=0 Centroidz=715.25
Frame=E22 JointI=22 JointJ=23 IsCurved=No Length=52.51 Centroidx=1143.945
Centroidy=0 Centroidz=754

TABLE:  "JOINT CONSTRAINT ASSIGNMENTS"
Joint=3 Constraint=BODY1 Type=Body
Joint=3 Constraint=BODY4 Type=Body
Joint=5 Constraint=BODY1 Type=Body
Joint=8 Constraint=BODY4 Type=Body
Joint=10 Constraint=BODY2 Type=Body
Joint=10 Constraint=BODY5 Type=Body
Joint=12 Constraint=BODY2 Type=Body
Joint=17 Constraint=BODY3 Type=Body
Joint=17 Constraint=BODY6 Type=Body
Joint=19 Constraint=BODY3 Type=Body
Joint=23 Constraint=BODY7 Type=Body
Joint=26 Constraint=BODY7 Type=Body
Joint=28 Constraint=BODY4 Type=Body
Joint=28 Constraint=BODY5 Type=Body
Joint=29 Constraint=BODY4 Type=Body
Joint=29 Constraint=BODY6 Type=Body

TABLE:  "JOINT RESTRRAIN ASSIGNMENTS"
Joint=3 U1=No U2=No U3=Yes R1=No R2=No R3=No
Joint=8 U1=No U2=No U3=Yes R1=No R2=No R3=No
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Joint=17 U1=No U2=No U3=Yes R1=No R2=No R3=No
Joint=28 U1=No U2=No U3=Yes R1=No R2=No R3=No
Joint=29 U1=No U2=No U3=Yes R1=No R2=No R3=No
Joint=30 U1=Yes U2=Yes U3=Yes R1=No R2=No R3=No

TABLE:  "JOINT SPRING ASSIGNMENTS 1 - UNCOUPLED"
Joint=3 CoordSys=Local U1=0 U2=0 U3=0 R1=0 R2=15308275.8157 R3=0
Joint=10 CoordSys=Local U1=0 U2=0 U3=0 R1=0 R2=2642583.4042 R3=0
Joint=17  CoordSys=Local  U1=0  U2=0  U3=0  R1=0  R2=1824096.5163  R3=0

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<td>RelDist</td>
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<td>Gravity</td>
<td>RelDist</td>
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<td>0</td>
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<td>Gravity</td>
<td>RelDist</td>
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<td>0.0809</td>
</tr>
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<td>E9</td>
<td>SLAB_PORTAL_SIDE</td>
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<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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<tr>
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<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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<td>0</td>
<td>121.2028</td>
<td>0.08485</td>
<td>0.08485</td>
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</table>

TABLE: "FRAME LOADS - DISTRIBUTED"

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<tr>
<th>Frame</th>
<th>LoadCase</th>
<th>CoordSys</th>
<th>Type</th>
<th>Dir</th>
<th>DistType</th>
<th>RelDistA</th>
<th>AbsDistA</th>
<th>AbsDistB</th>
<th>FOverLA</th>
<th>FOverLB</th>
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<tr>
<td>E1</td>
<td>SLAB_PORTAL_SIDE</td>
<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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<td>0</td>
<td>99.4786</td>
<td>0.0809</td>
<td>0.0809</td>
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<tr>
<td>E6</td>
<td>SLAB_PORTAL_SIDE</td>
<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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<tr>
<td>E9</td>
<td>SLAB_PORTAL_SIDE</td>
<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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<td>0</td>
<td>121.2028</td>
<td>0.08485</td>
<td>0.08485</td>
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<tr>
<td>E11</td>
<td>SLAB_PORTAL_SIDE</td>
<td>GLOBAL</td>
<td>Force</td>
<td>Gravity</td>
<td>RelDist</td>
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</table>

167
Frame=E12  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=16.8629771985851
FOverLA=0.085  FOverLB=8.51833333333333E-02
Frame=E13  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=127.16622979392
FOverLA=8.51833333333333E-02  FOverLB=8.62916666666666E-02
Frame=E13  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=127.16622979392
FOverLA=8.51833333333333E-02  FOverLB=8.62916666666666E-02
Frame=E14  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=127.08327978141
FOverLA=8.62916666666666E-02  FOverLB=8.74333333333333E-02
Frame=E14  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=127.08327978141
FOverLA=8.62916666666666E-02  FOverLB=8.74333333333333E-02
Frame=E15  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=81.7535931932046
FOverLA=8.74333333333333E-02  FOverLB=8.81666666666666E-02
Frame=E15  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=81.7535931932046
FOverLA=8.74333333333333E-02  FOverLB=8.81666666666666E-02
Frame=E16  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=81.7023867460431
FOverLA=8.81666666666666E-02  FOverLB=0.0889
Frame=E16  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=81.7023867460431
FOverLA=8.81666666666666E-02  FOverLB=0.0889
Frame=E18  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=16.5198365609349
FOverLA=0.0889  FOverLB=8.90416666666666E-02
Frame=E18  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=16.5198365609349
FOverLA=0.0889  FOverLB=8.90416666666666E-02
Frame=E19  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=52.51
FOverLA=6.51833333333333E-02  FOverLB=6.51833333333333E-02
Frame=E19  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=52.51
FOverLA=6.51833333333333E-02  FOverLB=6.51833333333333E-02
Frame=E20  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=121.4802866666
FOverLA=9.02833333333333E-02  FOverLB=121.48028666666
Frame=E20  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=121.4802866666
FOverLA=9.02833333333333E-02  FOverLB=121.48028666666
Frame=E21  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=130.89839609407
FOverLA=9.02833333333333E-02  FOverLB=130.89839609407
Frame=E21  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=130.89839609407
FOverLA=9.02833333333333E-02  FOverLB=130.89839609407
Frame=E22  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=52.51
FOverLA=6.51666666666666E-02  FOverLB=6.51666666666666E-02
Frame=E22  LoadCase=SLAB_PORTAL_SIDE  CoordSys=GLOBAL  Type=Force  Dir=Gravity
DistType=RelDist  RelDistA=0  RelDistB=1  AbsDistA=0  AbsDistB=52.51
FOverLA=6.51666666666666E-02  FOverLB=6.51666666666666E-02

TABLE: "FRAME DESIGN PROCEDURES"
Frame=E1  DesignProc="From Material"
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Frame=E16 DesignProc="No Design"
Frame=E17 DesignProc="No Design"
Frame=E18 DesignProc="No Design"
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Frame=E20 DesignProc="No Design"
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Frame=E22 DesignProc="No Design"

TABLE: "OVERWRITES - STEEL DESIGN - AISC-LRFD93"
Frame=6 DesignSect="Program Determined" FrameType="Program Determined" Fy=0 RLLF=0 AreaRatio=0 XLMajor=0 XLMinor=0 XKMajor=0 XKMinor=0 CmMajor=0 CmMinor=0 CB=0 BMajor=0 BMajor=0 BMajor=0 BMajor=0 PhiPnc=0 PhiPnt=0 PhiMn3=0 PhiMn2=0 PhiMn2=0 PhiMn3=0 CheckDefl="Program Determined" DeflType="Program Determined" DLRat=0 SDLAndLLRat=0 LLRat=0 TotalRat=0 NetRat=0 DLAbs=0 SDLAndLLAbs=0 LLAbs=0 TotalAbs=0 NetAbs=0 SpecCamber=0
Frame=7 DesignSect="Program Determined" FrameType="Program Determined" Fy=0 RLLF=0 AreaRatio=0 XLMajor=0 XLMinor=0 XKMajor=0 XKMinor=0 CmMajor=0 CmMinor=0 CB=0 BMajor=0 BMajor=0 BMajor=0 BMajor=0 PhiPnc=0 PhiPnt=0 PhiMn3=0 PhiMn2=0 PhiMn2=0 PhiMn3=0 CheckDefl="Program Determined" DeflType="Program Determined" DLRat=0 SDLAndLLRat=0 LLRat=0 TotalRat=0 NetRat=0 DLAbs=0 SDLAndLLAbs=0 LLAbs=0 TotalAbs=0 NetAbs=0 SpecCamber=0
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TABLE: "OVERWRITES - CONCRETE DESIGN - ACI 318-05/IBC 2003"
Frame=1 DesignSect="Program Determined" FrameType="Program Determined" RLLF=0 XLMajor=0 XLMinor=0
Frame=2 DesignSect="Program Determined" FrameType="Program Determined" RLLF=0 XLMajor=0 XLMinor=0
Frame=E1A DesignSect="Program Determined" FrameType="Program Determined" RLLF=0 XLMajor=0 XLMinor=0

END TABLE DATA
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