Final Research Report

Theoretical Model for Damage and Vibration Response in Concrete Bridges

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Abstract

The use of vibration signatures for structural health monitoring (SHM) purposes has been used in various fields, such as mechanical and aerospace engineering for many years. In recent years, its potential for use with civil engineering structures has been investigated and of particular interest in civil engineering is its applicability to buildings and bridges. It is recently known that each structure has its typical dynamic behaviour, which may be addressed as vibrational signature. Any changes in a structure, such as all kinds of damages and deteriorations leading to decrease of the load-carrying capacity have an impact on dynamic response, hence suggesting the use of dynamic response characteristics for the evaluation of quality and structural integrity. Monitoring of the dynamic response of structures makes it possible to get very quick knowledge of the actual conditions and helps in planning of rehabilitation budgets. One of the promising developments in structural vibration monitoring is the ambient vibration testing which does not require a controlled excitation of the structure. The structural response to ambient excitation can be recorded in large number of points and from these ambient measurement, the condition of the structure can be derived. A classification of the structures can be developed based on vibration monitoring using the modal parameters natural frequencies, mode shapes, damping values and vibration intensities. The ambient vibration testing represents a real operating condition of the structure.

This report presents a theoretical and experimental ambient modal analysis on three existing structures namely a staircase, a timber footbridge and a concrete bridge. The field-testing has provided opportunity to analyse dynamic properties of the three selected structures. The operational modal analysis software, ARTeMIS Extractor is a tool used for analysing the raw data to obtain the dynamic properties of the structures. Finite element modelling and analysis on the structure by using finite element software, ANSYS were developed. The comparison between the mode shapes determined from both analyses showed some similarity. The natural frequencies that were generated had a variance between the two analyses. Thus, the modal updating is essential on the next stage. Improvement in the field-testing is needed in order to obtain more accurate and quality results. Overall, modal analysis is comparable as an alternative to extract dynamic properties of the structures.

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Model Teoritikal bagi Kerosakan dan Respon Getaran dalam Jambatan Konkrit

Penggunaan kaedah 'tanda tangan' getaran untuk tujuan penyeliaan keadaan kesejahteraan struktur telah dibgunakan dalam pelbagai bidang seperti kejuruteraan mekanikal dan aero-angkasa sejak beberapa tahun dahulu. Kebelakangan ini, potensi penggunaan kaedah ini dalam struktur kejuruteraan awam telah dikaji terutamanya applikasinya dalam penyelenggaraan dan penyeliaan bangunan dan jambatan. Adalah dikertahui bahawa sesebuah struktur mempunyai kelakuan dinamik tertentu, yang boleh disifatkan sebagai 'tanda tangan getaran'. Sebarang perubahan di dalam struktur berkenaan, seperti pelbagai kerosakan dan keusangan yang mengakibatkan pengurangan keupayaan menanggung beban, akan memberi kesan ke atas respon dinamik. Oleh yang demikian, ianya mengisyaratkan bahawa ciri-ciri respon dinamik boleh digunakan untuk penilaian kualiti dan integriti sesebuah struktur. Salah satu perkembangan yang berpotensi dalam bidang penyeliaan getaran struktur ialah teknik pengujian getaran sekitar yang tidak memerlukan penggegaran terkawal ke atas struktur yang diuji. Respon struktur terhadap getaran sekitar boleh direkodkan dalam poin yang banyak dan maklumat keadaan struktur boleh diperolehi dari pengukuran tersebut. Satu sistem klsifikasi juga boleh diwujudkan berdasarkan parameter modal gelombang semula jadi, bentuk modal, nilai penyusutan dan intensiti getaran. Pengujian getaran sekitar memberikan keadaan operasi sebenar sesebuah struktur.

Laporan ini menyampaikan analisis modal dengan getaran sekitar secara teori dan eksperimen pada tiga struktur yang sedia ada iaitu sebuah struktur tangga, sebuah jambatan kayu dan sebuah jambatan. Ujikaji di lapangan telah menyediakan peluang untuk menganalisis ciri-ciri dinamik tiga struktur yang terpilih. Perisian analisis getaran sekitar, ARTeMIS Extractor digunakan untuk menganalisis data bagi mendapatkan ciri-ciri dinamik struktur tersebut. Model telah dibina dan dianalisis dengan menggunakan perisian kaedah unsur terhingga, ANSYS. Perbandingan dari kedua-dua analisis telah menunjukkan bahawa terdapat kesamaan dari segi bentuk modal. Terdapat perbezaan dalam frekuensi semula jadi yang dihasilkan dari kedua-dua analisis. Justeru, pengemaskinian modal adalah penting dan diperlukan untuk tahap yang seterusnya. Peningkatan prestasi pada ujian di lapangan diperlukan untuk mendapat keputusan yang lebih tepat dan berkualiti. Secara keseluruhannya, analisis modal merupakan suatu cara yang sesuai dan praktikal bagi mendapatkan ciri-ciri dinamik struktur.

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1 Introduction

1.1 General Introduction

Vibrations or dynamic motions are naturally to life and regarded by mankind as unpleasant and unwanted phenomena causing undesirable consequences such as discomfort, noise, malfunctioning, fatigue, destruction and collapse. Structures being systems of elastic components receive response, which subjected to dynamics and vibration analysis from external and internal forces with finite deformations and overall motion. Natural disasters such as earthquakes are the most frightening manifestations of dynamic motion, which have large destructive impact on man-made structures (Nuno M. M. Maia and Julio M. M. Silva, 1997).

Vibration is an element, which is hard to avoid in practice. Excitation of resonant frequencies of some structural parts can occur with existence of vibration even it is only a small insignificant vibration. Then it can be amplified into major vibration and noise sources. Vibration can be easily defined as an oscillation which is the analogous to the motion of the particles of a mass of air or the like, whose state of equilibrium has been disturbed. It exhibited a movement first in one direction and then back again in the opposite direction. The number of times that a complete motion takes place during the period of one second is called frequency which is measured in Hertz (Hz).

According to Bruel & Kjaer, oscillation normally varies with time and magnitude of a quantity refers to an indication, which may be greater or smaller than that indication. Basically random vibration is described by the three factors, which are amplitude, size and frequency. Nowadays, vibration measurement and analysis on civil engineering structures has begun to gain popularity in civil engineering field. Technology developments have created an increasing requirement for reliable dynamic analysis.

In civil engineering field, the behaviour of a structure at "resonance" is a key aspect of structural dynamic analysis. The natural frequency of vibration of a structure corresponds to that structure's resonant frequency. Maximum displacements are produced if a structure is subjected to vibration at its natural frequency. The stresses, which developed in the framing members and connections of the structure, are greater when the displacements are increasing. For each mode, the structures will vibrate with a particular distorted shape called mode shape. After that the vibration dies out because of damping that removes energy from the structure. The interest of human’s ability to monitor a structure and detect damage at the earliest stage is persistent throughout the civil engineering communities. The damage detection methods used are either visual or localized such as magnet field methods, acoustic and ultrasonic methods, thermal field methods, radiography and eddy-currents methods. However most of these methods are subjected to limitations that require area of damage which known as a priori and is readily accessible. Only the damage near the surface can be detected through these methods (Charles R.Farrar and Scott W.Doebling, 1997).

Therefore there is a need to have a strong reliable vibration analysis tool to provide understanding of structural characteristics, operating condition and performance criteria that enable designing optimal dynamic behaviour or solving structural dynamic problems in existing designs (Nuno M. M. Maia and Julio M. M. Silva, 1997). Modal analysis is an approach of knowing the natural frequency, mode shape and damping properties for a certain structure. According to D.J Erwins, modal testing is the process involved in testing components or structures with the purpose to obtain a mathematical description of their dynamic or vibration behaviour. Indirectly it can provide the new knowledge and experience of civil engineering, which can be used for future generic design. Most recently, development and verification of modal models have been of higher concern in today’s world. It is an approach for structural modification based on modal analysis results.

There are mainly two types of structural dynamic testing namely forced vibration testing and ambient vibration testing. The force method is conducted by dropping a known valued force on the structure, which will induce a condition of free vibration. The ambient vibration testing represents a real operating condition of the structure, which uses the disturbances, induced by traffic and wind as natural or environmental excitations.

Modal analysis has become a major alternative to provide a helpful contribution in understanding control of many vibration phenomena, which encountered in practice. Determining the nature and extent of vibration response levels and verifying theoretical models and prediction are both major objectives that can be achieved with experimental modal testing. Structures all over the world have become study subjects for the testing. First to emerge is the three spans, Z24 Bridge at Switzerland where a few of system identification analysis were performed to determine the dynamic characteristics of the bridge.

Repair and maintenance of infrastructure facilities are rapidly becoming a major financial burden for authorities bringing forth many new challenges for civil engineers. Key to the successful upgrading of such structures is timely detection and quantification of damage and deterioration, and in particular those, which builds-up over time during the functional life of the structure. The large stock of buildings and bridges, which are suffering from damage and deterioration, would present a set of new challenges in the management of maintenance issues. Factors such as improper design and construction without regard for maintainability or life cycle costs, coupled with poor performance reliability and efficiency, would incur high future maintenance costs. This project attempts to identify the potential maintenance problems and maintenance management issues and propose a realistic long-term approach for effective structural health monitoring. This research looks into the potential use of vibration signatures for structural health monitoring purposes which has been used in various fields such as mechanical and aerospace engineering, in the assessment and monitoring of civil engineering structures. Of particular interest in civil engineering is the applicability of dynamic response characteristics for the evaluation of quality and structural integrity of buildings and bridges. Monitoring of the dynamic response of these structures makes it possible to get information of the actual conditions and helps in planning of rehabilitation budgets. The research will look into the development of a structural classification system for civil engineering structures, based on ambient vibration monitoring using modal parameters including natural frequencies, mode shapes, damping values and vibration intensities.

The proposed classification based upon vibration signatures would be useful in providing a baseline data and can be used to identify structures that show distinct problems and require urgent maintenance and rehabilitation actions. Such classification system can be an effective tool for assessment and priority ranking of the structures where a proposed budget planning can be done according to the time schedule set up based upon the measured results of the ambient vibration signatures.

Basically structural evaluation is based almost entirely on visual inspection, which is limited by accessibility and subjectivity. It is normally carried out when there is a segment of the structure suspected to be defective. Structures assessment in health monitoring also can be carried out based on the modal properties obtained from the structures. Besides that, most of the structures in Malaysia are designed based on static load approach without consideration of vibration on the structure during the design stage. Vibration design is under consideration mostly in the high rise building design only. Malaysia is not located in the earthquake and typhoon zone but the incidents that happen lately such as Tsunami and earthquake in South East Asia region had a slightly impact on Malaysia. Some parts of Malaysia can feel the vibration of the earthquake when it happened outside of Malaysia.

Modal analysis is a solution to obtain the actual structural dynamic properties. The dynamic properties, which consist of natural frequency, mode shape and damping, are unknown on the design. The frequencies of vibration of the structure are directly related to the stiffness and the mass of structure while the mode shapes are related to the defect location. Therefore vibration testing needs to be carried out to obtain the data of those dynamic properties for structural health monitoring and evaluation.

Civil engineering structures normally are in large size so it is viable to get excitation from ambient vibration method. Ambient testing is chosen for this study due to its importance in representing the real operating condition that could be related to normal excitation or natural disaster especially impact of excessive wind load on structure. Since ambient testing does not interrupt service of the test structure, it can be conveniently applied for long term health monitoring of structures.

1.2 Problem Statement

Recent issues on problems in structural integrity of buildings and bridges in Malaysia has highlighted the challenges faced by the authorities who need effective methods in structural priority ranking concerning maintenance, rehabilitation planning and structural monitoring actions. Buidling or infrastructure owners and authorities, who need to manage a large stock of structures will face challenges when structural performance falls short of “excellent” because the serviceability and economic impacts are too severe to accept. Owners of buildings and other civil infrastructure require effective methods in structural priority ranking concerning actions on maintenance, rehabilitation planning and lifetime assessment. With limited maintenance budgets, managers and owners require priority-ranking tool in decision making to link between optimized use of budget and effective maintenance activities.

The use of vibration signatures for structural health monitoring (SHM) purposes has been deployed in various fields, such as mechanical and aerospace engineering for many years. In recent years, its potential for use with civil engineering structures has been investigated and of particular interest in civil engineering is its applicability to buildings and bridges. It is recently known that each structure has its typical dynamic behaviour, which may be addressed as vibrational signature. Any changes in a structure, such as all kinds of damages and deteriorations leading to decrease of the load-carrying capacity have an impact on dynamic response, hence suggesting the use of dynamic response characteristics for the evaluation of quality and structural integrity. Monitoring of the dynamic response of structures makes it possible to get very quick knowledge of the actual conditions and helps in planning of rehabilitation budgets.

One of the promising developments in structural vibration monitoring is the ambient vibration testing which does not require a controlled excitation of the structure. The structural response to ambient excitation can be recorded in large number of points and from these ambient measurement, the condition of the structure can be derived. A classification of the structures can be developed based on vibration monitoring using the modal parameters natural frequencies, mode shapes, damping values and vibration intensities. The proposed classification can be used to identify structures, which show distinct problems and urgently require maintenance and rehabilitation actions. A proper budget planning of the responsible authority can be done according to the time schedule set up based upon the measured results of the ambient vibration system. Such classification can be a very effective tool for assessment and priority ranking of the structures.

1.3 Objectives

The purpose of this research is to study the dynamic properties and behaviour of selected structures by using ambient modal analysis and compare with the finite element analysis. Objectives of this research are as follow:

a) Identify the parameters and criteria for determining vibration signature of bridge structures
b) Obtain the experimental dynamic properties, which consist of natural frequency, mode shape and damping ratio of selected structures using operational modal analysis.
c) Develop a theoretical finite element model for correlating the severity of damage and the change in resonant frequency and damping of selected structures.
d) Perform modal analysis of the selected structures to obtain the theoretical dynamic properties, which consist of natural frequency and mode shape.
e) Compare measured results from ambient vibration analysis of the tested structures with the finite element analysis of the structures.

1.4 Scope of Study

The scope of research that needs to be carried out consists of two parts, which are theoretical part and experimental vibration testing. First part is the experimental modal analysis on selected structures and the analysis are done using software called ARTeMIS Extractor. The ambient experimental testing is conducted using natural sources and hammer approach as artificial impact. A trial experimental testing is conducted on the first stage of experimental testing. The small laboratory trial experimental testing on a plate is conducted at the Vibration Laboratory.

Three structures in Universiti Teknologi Malaysia (UTM) are selected to conduct modal analysis as follows:

1. Preliminary vibration test on a staircase structure
2. Vibration test on a timber footbridge.
3. Full-scale vibration test on a concrete bridge.

It is followed by the second part, which is theoretical part, involving the use of finite modelling. ANSYS software is used in the modelling and analysis for the selected structures. The results from the analysis in ANSYS will be the theoretical dynamic properties of the structures. Lastly, comparison is made between the experimental results and the finite element analysis to extract similar dynamic properties of the tested structures.

2 Literature Review

2.1 Introduction to Vibration

In Civil Engineering field, vibration has become a big concern for civil engineers nowadays in their design. Before that, civil engineers pay more attention to static analysis in their design but some incidents happen where one of the famous oscillation incidents was the great Tacoma Narrow Bridge in Washington State. A steady wind led the bridge to ultimate destruction of this fine structure only few months after the completion. It became the object of scrutiny for structural engineer and none of them want to repeat the costly mistake in future.

Vibrations or oscillations can be regarded as a solution of dynamics in which when subjected to internal or external restoring forces either due to elasticity or gravity, a system swing back and forth about an equilibrium position and is defined as an assemblage of parts acting together as a whole. There is one dangerous phenomenon where the loading imposed on the structure would create a periodicity that synchronises with the natural frequency of the structure (J.R Maguire and T.A. Wyatt, 1999). In another word, the force will cause a perceptible motion if the frequency of application is well below the vibrating structure's frequency that can oscillate freely with distortion on its own (R.E.D Bishop, 1979). This magnification is known as resonance, which will create exaggeration of response with potential fatigue failure that needs to be avoided in civil engineering.

2.2 Basic Concepts

2.2.1 Dynamic Loading

Based on the nature of the time-wise variation, dynamic loading can be categorized into four types, which are (J.R Maguire and T.A. Wyatt, 1999):

a. Harmonic or periodic.
   It is also known as a steady state loading which frequency and amplitude play the major role and time playing for a secondary role only. The loading consist forces that simply proportional to the trigonometric function of sin ωt or cos ωt or a combination of both. Amplitude of this load type repeats itself many times in a regular basis. The common characteristic of the function from the loading is its values can be determined for any future time t, which is called deterministic. Machinery loading is the example of harmonic loading.

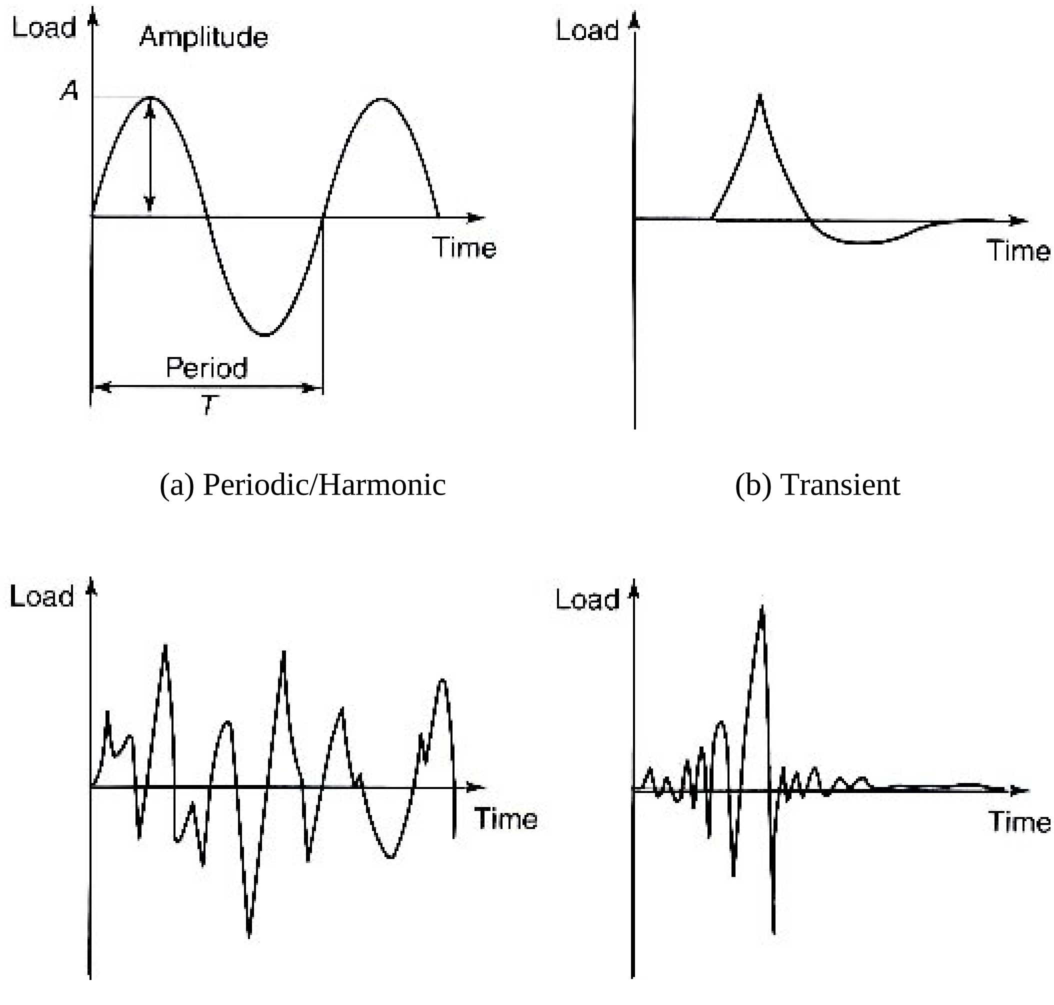
b. Transient.
   Initial excitation is referred to at times as transient excitation where all motions caused by initial excitations come to rest eventually. Transient loading varies with time so it does not repeat itself continuously. It is exactly contrast with harmonic loading. Blast loading is one of the examples in this category.

c. Stationary random.
   This type of loading is in the category where the phenomena of the outcome at a future instant of time cannot be predicted. Although the value of the load cannot be determined

precisely, the statistical properties of the loading vary in a gradual manner such as wind loading that varies in velocity so as the excitation imposed.

d. Non-stationary random.
   It is actually the same as stationary random but the difference is the statistical properties vary only swiftly. Earthquake loading is categorized in this type of loading because earthquake frequently happens in a flash of time.

The Figure 2.1 illustrates the four types of main loading and they are plot of loads against time, known as the time-domain representation. However, the amplitude against frequency function can be plotted which is known as frequency domain representation. Frequency-domain often used to highlight the most significant characteristics of the time-domain obscuration loading. The frequency domain equivalent of Figure 2.1 is shown in Figure 2.2



(a) Periodic/Harmonic (b) Transient

(c) Stationary Random (d) Non-Stationary Random

Figure 2.1: Examples of dynamic loading (time-domain representation)

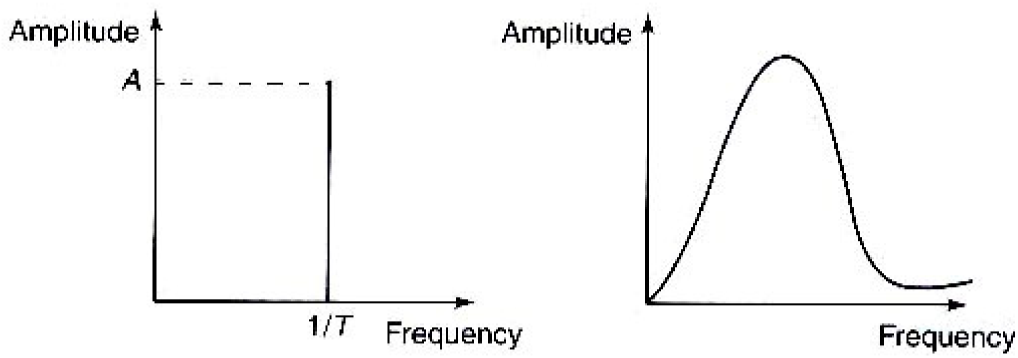


Figure 2.2: The frequency-domain equivalent

2.2.2 The Dynamic System

The dynamic system in civil engineering field can be divided according to their behaviour into two major types. Implication of the principle of superposition is usually valid in civil engineering structure and it is called linear system. However, the principle of superposition that linear system holds cannot be applied on the nonlinear system. The distinction between linear and non-linear systems frequently depends on the range of operation but not inherent property of the system. Although there are many approaches to study non-linear system, qualitative and quantitative approaches are most commonly used.

Linear analysis is more familiar in dynamic because it is to assist evaluation by modal analysis. Another reason that linear analysis remains common is that it is suitable for many dynamic criteria such as fatigue and comfort, and the dynamic response is mostly within the limit. Therefore, any system must have certain characteristics before it will vibrate. In first Newton's Law, it will stay in the equilibrium position and will tries to return to the stable position if it is disturbed. The restoring force is called stiffness force, which is proportional to the displacement of the structure while the coefficient of proportionality is called the stiffness of the structure. The stiffness possesses potential energy or strain energy within the structure. Requirement to overshoot its equilibrium position is the structural must also own mass to generate vibration.

After the certain period, the motion starts to slow down and stop after awhile.
In this case, the system is said to be 'damped'. Some energy dissipating mechanism is behind this phenomenon. The damping will cause the amplitude of the displacements to fade with time if there is no additional energy input. An ideal assumption is made where the damping force is proportional to the velocity of the structure. In conclusion, a common idealization is to assume that the damping is vicious.

2.2.3 Equation of Motion

Basically, the inertia force, damping force and stiffness force together with externally applied force will form the equation of equilibrium between them, which is called the 'equation of motion' that defines the dynamic behaviour of the structure.
It is in the form of:

Inertia force + damping force + stiffness force = external force

When in algebraic form, the equation becomes: $m\ddot{y} + c\acute{y} + ky = f\,(t)$

The equation of motion controls almost any structure's linear dynamic behaviour and the dynamic response can be found by solving this equation of motion. Values at the selected location, the degrees of freedom (DOF) will embody the displacement in a structure. Figure 2.3 shows the example of single DOF system. The reason it is chosen because an accurate estimation of the kinetic energy can be made. The equation is form in matrix form where the stiffness is called K, the mass matrix is called M and the damping matrix is called C and the equation is:

$$M\ddot{y} + Cy + Ky = F$$

Figure 2.3: Example of single DOF system

2.2.4 Modes of Vibration

Every structure does not necessary has only single degree of freedom (DOF) but the system will have numbers degree of freedom and thus it will have numbers of solution indicating the modes of vibration of the system. A free vibration exists in each mode with certain frequency and certain mode shape.

The normal modes are independent of each other where they do not affect each other's mode. The problem can be solved by using the simple degree of freedom solution' to obtain the superposition of the modal response. This is where the modal analysis plays its extremely powerful part and adopted almost universally.

2.3 Sources of Vibration

Vibration and shock can arise from a variety of sources, which may be generated from natural sources and also possibly from human activities. They may produce annoyance and discomfort. Some of them even involve structural damage when severe conditions occur and the extensive damages may cause disasters that are beyond our imagination. The typical sources of vibration either natural or excitations from human activities are:

a. Road vehicles
b. Earthquakes
c. Aircraft
d. Machinery and industrial plant
e. Forging hammers and drop-stamps
f. Pile driving
g. Wind
h. Hydrodynamic loading
i. Blasting

Civil engineer should not overlook on the vibration that occur on a structural that may turn into unstoppable disaster. Therefore as a civil engineer, one should be able to determine the sources of vibration and encounter them in the structural design.

2.4 Modal Analysis

Structural health monitoring is pervasive in civil engineering because it is an analysis of the dynamic behaviour of civil structures to observe and examine the integrity of those structures. It involves different parameter and damage identification. Visual inspection is limited by the inability to classify invisible damage, which only can detect the damage near the surface. However, modal analysis fulfils the requirements of global in nature and automated that examine changes in the vibration characteristics of the structure (Ayman Khalil, Lowell.G, Terry J.W and Douglas W., 1998).

Modal analysis was first applied around 1940 as an engineering tool in the search for better understanding of aircraft dynamic behaviour. However, it did not see extensive development in scientific and engineering until availability of the invention of smaller, faster computer and boosting in signal analysis with the introduction of signal processing algorithm such as Fast Fourier Transform (FFT) spectrum analysers (Nuno M. M. Maia and hulio M. M. Silva. 1997). Modal testing become a mature technology in the 1980s and advanced to feature in mechanical and aerospace Master of Science and undergraduate curricular at universities in Europe and North America. Although modal testing was developing in 1990s, it still remained unfamiliar research topic in civil structural engineering. Then finite element model updating based on modal testing results was under great development in 1990s (Aleksandar Pavic 1999). Today, modal analyses are used on large engineering structures, which are subjected to dynamic motions or vibrations.

It is well known that structures can resonate where the small forces can result in important deformation and damage can be induced in the structure. For an example, a bridge disaster where the Tacoma Narrows suspension bridge collapsed due to wind-induced vibration on November 7, 1940. The bridge was only opened for traffic just a few months before it collapsed. In addition, most of the structures can be made to resonate. Interaction between the inertial and elastic properties of the materials within a structure contributes to resonant vibration. Identification and quantification of resonant frequencies of a structure is needed to obtain better understanding of any structural vibration problem. That is the reason why today modal analysis is developed and used to access structural dynamic behaviour of certain structures (Patrick Guillaume, 2002).

2.4.1 Function of Modal Analysis

The main function of modal analysis is the process to describe the dynamic behaviour of a structure from test data construction to a mathematical model. According to Nuno M. and Julio M., a set of accurate and sufficiently extensive in both the frequency and spatial domain Frequency Function (FRFs) are able to acquire from this form of vibration testing. Then, analysis and extraction of the properties for all the required modes of the structure can be done from the FRFs (Nuno M. M. Maia and hulio M. M. Silva, 1997). Natural frequencies, mode shapes and modal damping ratios are the measurement and estimation of a structure's modal properties. They serve as parameters that detect and locate possible damage of structures, long-term health monitoring of structures, basis input to the finite element model updating and evaluation of structures against different harsh situation such as wind load and earthquake.

Modal analysis is capable of extracting the modal natural of the structures from the measurement made on the vibrating structures with a range of analysis procedures. The independence of modal properties serve as the basis of a mathematical model of the same structure which leads to the same modal properties starting not from data that measured, but from the mass, damping and stiffness distribution that assumed in the structure (Aleksandar Pavic 1999).

The capabilities of the transducer and data processing equipment will affect frequency domain extent of the model during the testing while the experimental setup will influence the accuracy and correctness of the model. There will be unwanted modification effect on the structure for almost all the methods of applying as well as unwanted influence from almost all the response measurement transducers and support fixture on the structure. Awareness should be taken care to select the appropriate method of excitation and response measurement for obtaining the accurate and reliable results (Nuno M. M. Maia and hulio M. M. Silva. 1997).

2.4.2 Routes to Modal Analysis

Modal analysis can be carry out in experimental and theoretical. There are three key models that can be used to illustrate dynamics of a vibrating structure, which are:

1. The modal model
The structure's dynamic properties are represented in terms of natural frequencies, mode shapes and modal damping ratios.

2. The spatial model
The structure's dynamic properties are represented in terms of its mass, stiffness and damping properties

3. The modal model
The model represents the structure's dynamic properties in terms of series of transfer functions, usually the Frequency Response Functions (FRFs).

2.4.3 Experimental Modal Analysis

Experimental modal analysis is the process that applies experimental approach to determine the modal parameters of a linear, time invariant system. The modal parameters describe most of the vibration and acoustic problems for both the functions and the system characteristics. There are several simple and short phases involve in the process of determining modal parameters. The process of determining modal parameters from experimental data involves several phases. Specific goals in terms of reducing the errors associate with that phase will determine the success of the experimental modal analysis. The possible delineation of the phases are:

a) Modal Analysis Theory
b) Experimental Modal Analysis Methods
c) Modal Data Acquisition
d) Modal Parameter Estimation
e) Modal Data Presentation/Validation

Assumptions concerning any structure are made in order to perform an experimental modal analysis. However, these basic assumptions are only approximately true and they can be evaluated experimentally during the test and after performing the data analysis. The validity of the assumptions involved in modal analysis need to be measured before conducting the testing. The basic five assumptions that usually made are:

1. The structure is assumed to be linear
The structure obeys the principle of superposition where the response of the structure to any combination of forces that applied simultaneously is the sum of the individual responses to each of the forces acting alone. A controlled excitation experiment that apply the forces applied to the structure with a form convenient for measurement and parameter estimation which relatively almost similar to the forces that act naturally to the structure in its normal environment can characterise the structure's behaviour.

2. The structure is time invariant
Time invariant means that the parameters that are to be determined are constants throughout the testing. Components such as mass, stiffness, or damping on a system, which is not time invariant generally depend on factors that are not measured or are not included in the model.

3. The structure obeys Maxwell's reciprocity
It means that the frequency response function between a point called p and another point called q is the same when the response on a point called p when force is excited on a point called q compared with the measured response at q when a force applied at p ($Hpq = Hqp$). Another explanation for reciprocity is that the degree-of-freedom *p* causes a response at degree-of-freedom *q* that is the same as the response at degree-of-freedom *p* caused by the same force applied at degree-of freedom *q*.

4. The structure is observable
Adequate behavioural model of the structure can be generated with sufficient information from the input-output measurements. Structures are not complete observable due to the loose components or degrees-of freedom of motion that are not measured. This occurs in at least two different ways. The first way is the limitation of data to a minimum and maximum

frequency and limited frequency resolution. Secondly, the information relative to local rotations is not available because lack of transducers available in this area.

5. The structure is causal and stable
The system is in rest mode before it is excited if the structure is causal while the structure is stable because the vibrations will cease after the excitation is removed.

2.4.4 Frequency Response Function

Frequency Response Function (FRF) consists of real and imaginary components and it acts as a complex transfer function, which expressed in the frequency domain. It may also be represented in terms of magnitude and phase where the response parameter may appear in denominator or numerator of the transfer function. It displays the independent of the excitation function characteristic where the excitation could be random, periodic or transient function of time. Furthermore measured data or analytical functions form a frequency function expresses the structural response as a function of frequency in terms of displacement, velocity, or acceleration from the result obtained with one type of excitation that can be used to predict the response of the structure to any other type of excitation (T. Irvine 1999).

2.4.5 Frequency Response Function Model

Frequency Response function are properties of linear dynamic systems.
Figure 2.5 shows a linear system where F(□is the input force of the angular frequency (□□□is the transfer function while X(□) is the displacement response.

They are complex and represented in terms of magnitude and phase. Each of them is Fourier transform, which has its own function.

Figure 2.5: Linear system

Therefore, the equation that represents the relationship between input force, transfer function and displacement response is:

2.4.6 Modal Testing Mechanism

There is a variety of hardware component available for modal testing but there are main measurement mechanisms which are:

1. The excitation mechanism
2. The sensing mechanism
3. The data acquisition and processing mechanism

2.4.6.1 The Excitation Mechanism

In the excitation mechanism, the input under the form of driving force is applied at a given coordinate of the structure under analysis (Nuno M. M. Maia and Hulio M. M. Silva. 1997). There are two types of techniques generally used for modal test excitation which are impulse or impact hammer excitation and shaker excitation.

The first of these involves either out of contact throughout the vibration or only applied in contact for a short period. The second type includes an exciter which remains attached to the structure throughout the test with continuous or transient excitation (D.J Erwins, 1985). Each of the technique has their own features, advantages and disadvantages but data collected from both techniques is exactly the same from a theoretical standpoint. Difference between the data from practical point is caused by factors such as the structure suspension, the mass of the mounted transducer and stiffening effects of the shaker arrangement (Pete Avitable 1998). Impulse or impact hammer is simply a hammer with a force transducer attached to its head to determine component or system response to impacts of varying amplitude and duration by pairing with an accelerometer or more on the component being tested. It does not need a signal generator and power amplifies.

Force, pulse duration and frequency range are the most critical specifications for impulse hammer in modal testing. Hammer mass and hardness of its impacting head contribute to the difference of the range of frequency. Besides that, the amplitude of impact force is affected by the mass together with the velocity of impaction (Nuno M.
M. Maia and hulio M. M. Silva. 1997). It is the most commonly used technique that excites wide range of frequency in modal testing. Impulse hammer excitation is chosen because of the inexpensive cost, quick and easiness to perform the modal testing. However, sometimes the input force can be different from measurement to measurement which caused by lack of control over the frequency content of the excitation and poor signal ratio (Pete Avitable 1998). Figure 2.6 shows the hammer that induced force in modal testing.

Figure 2.6: Impulse hammer used in modal testing

Shaker excitation is mostly used in more complex structure with vibration provided by force generators or transducers. “Shaker is usually an electromagnetic or Electro-hydraulic vibrator driven by power amplifier (Nuno M. M. Maia and Hulio M. M. Silva. 1997). The measurement for shaker excitation is serial or parallel measurements. There are many types of excitation technique in shaker excitation and two that broadly used are steady state excitation and broadband excitation (P. Reynolds and A. Pavic 2000). Both frequency and amplitude can be easily controlled so it offers accurate result (Nuno M. M. Maia and Hulio M. M. Silva. 1997). Shaker excitation is suitable to use in repeatable experiment compare to impulse hammer excitation. It can be used for MIMO analysis, which has many inputs and outputs where data collected will be more accurate to produce good results. This technique is not widely used because of the difficult setup process and skill operators are needed to use the shaker excitation. It involves more equipment and channel that make it more costly to use in modal testing (P. Reynolds and A. Pavic 2000). Figure 2.7 shows the one of the example of shaker used in modal testing.

Figure 2.7: Shaker used in modal testing

2.4.6.2 The Sensing Mechanism

Basically, the sensing mechanism is constituted by sensing devices and conditioning amplifiers. Various sensing devices known as transducer are available in modal testing but piezoelectric transducers are usually used in experimental modal analysis. They are able to determine force excitation, which called forced transducers and determining response which

called accelerometers. Purpose of transducers is generation of electric signals that are proportional to the physical parameter which one wants to measure. Conditioning amplifier serve the purpose to solve the problem of weak signals generated and electric impedance mismatched (Nuno M. M. Maia and Hulio M. M. Silva. 1997). The examples of accelerometer are shown in Figure
2.8.

Figure 2.8: Examples of accelerometer

2.4.6.3 The Data Acquisition and Processing Mechanism

The complicated devices known as analysers basically are used in data acquisition and processing mechanism to determine development of signals by the sensing mechanism and to ascertain the magnitude and phases of the excitation forces and responses. The common used analysers which called Spectrum Analysers of FFT Analysers are based on the Fast Fourier Transform (FFT) algorithm and provide direct measurement of the FRFs. Transducers generate the analogue time domain signals that will be converted into digital frequency domain information that can be processed with digital computer are done by the analysers. In other words, analysers received measurements of time varying excitation forces applied to the structure and corresponding dynamic acceleration responses to compute FRF accelerance data for a selected structure (Nuno M. M. Maia and hulio M. M. Silva. 1997). Figure 2.9 shows the signal analyser used in modal testing.

Figure 2.9: Signal Analyser in modal testing

2.4.7 Measurement of FRF Matrix

2.4.7.1 Single Input and Single Output Testing (SISO)

The SISO testing is conducted using one roving excitation and one accelerometer attached to a single DOF as the fix reference of the structure. The fix reference then receives excitation sequentially from the hammer. Reference DOF needs to be selected carefully because it contains information about all the mode shapes in the interested frequency range. The reference in a nodal position for any mode should be prohibited from selection because the node of mode is a location of zero response (Pete Avitabile 2002).

2.4.7.2 Single Input and Multiple Outputs Testing (SIMO)

There is another alternative for modal testing that is multiple references testing where two or more response accelerometers are fixed to the two or multiple references. Then, one roving exciting is performed. Tri-axial accelerometer is often used to capture movements in all directions simultaneously. This testing is used when single reference DOF which contain all mode of interest is impossible to find and multiple references are required. Another advantage of SIMO testing over SISO testing is it can detect repeated roots.

2.4.7.3 Multiple Input Multiple Outputs Testing (MIMO)

The structure is excited at two or more DOFs and the output is measured in two or more response DOFs. MIMO testing is usually conducted on complex structure, which have local mode that the reference DOF with modal deflection for all modes is not available. The multiple outputs are measured on the same time to give consistent data. Furthermore the MIMO testing has the following advantages:

a) Distribution of sufficient energy over the complete structure.
   Sufficient energy is needed to distribute over the structure, which is large and heavily damped.

b) Avoidance of non-linear behaviour.
   Distributing the energy over the structure using multi-point excitation reduces the force level at the different excitation DOFs thereby avoiding driving the structure into non-linear behaviour that would deteriorate the estimation of the FRFs.

c) Better simulation of real-life operation
   MIMO testing provides a better representation of the excitation forces that load the structure during real-life operation.

d) Reduced force drop-offs at resonance frequencies
   Multiple smaller shakers will lead to smaller drop-offs at resonance frequencies.

e) Reduced test time
   MIMO testing consumed less time on-structure by using multiple shakers and multiple references.

2.4.8 Ambient Vibration analysis

Ambient vibration analysis is when the modal properties are only identified from measured response. It can be called “output only identification” or “operational modal identification” because only response data are measured but the actual loading conditions are unknown. The terms “ambient identification” or “ambient response analysis” are often used. Besides that, it does not directly lend itself to FRFs calculations because the input forces are not measured. The main purpose of ambient vibration analysis is to determine the modal parameters from experimental data. The modal parameters are mode shapes which the way the structure moves at a certain resonance frequency, natural frequencies that represent the resonance frequencies and damping ratios which the degree to which the structure itself is able of damping out vibrations.

Excitation on large civil engineering structures by natural loads are often not easily controlled, for instance wind loads on building, wave loads on offshore structures, or traffic loads on bridges. Thus, it is an advantage to use output-only modal identification. The natural excitation is used as the excitation source instead of artificially exciting the structure and dealing with the natural excitation as an unwanted noise source.

Ambient vibration testing is generally preferred to non-destructive forced vibration measurement techniques for obtaining the modal parameters of large structures for many reasons. A structure can be adequately excited by wind, traffic, and human activities and the resulting motions can be readily measured with highly sensitive instruments. Consequently, the overall cost of the measurements conducted on a large structure is reduced (Ventura, C.E. and Horyna, T, 2000). There have been several output-only data modal parameter identification techniques available that were developed by different investigators for different uses such as: peak picking from the power spectroutput only modal parameter ideal densities auto regressive-moving average (ARMA) model based on discrete-time data natural excitation technique Output-only modal parameter identification of civil engineering structures 3 (NExT), and stochastic subspace identification (X.He, B.Moaveni, J.P.Conte and A.Elgamal, 2003). The advantages of the operational modal testing are:

- Testing is cheap and fast because excitation equipment is not needed.
  Therefore, the physical setup is very straightforward and fast.

- There are difficulties or impossibilities to test the structures by forces.
  Artificial inputs cannot be applied to a structure or they cannot be measured correctly due to the structure's boundary conditions or physical size thereby making classical modal analysis impractical. Special boundary conditions are not needed for the testing and it is in-situ testing (Proefschrift, 2005).

- Measurements are done in the actual operation conditions for the structure but they do not interfere with the operation of the structure. Identified modal parameters represent the dynamic behaviour of the structure in reality.

There are also some disadvantages of conducting the modal operational testing which are:

a) The modal model used is not up to scale.
b) An assumption is made that excitation cover interested frequency range.
c) The calculation and computation is complicated and intensive which lead to time consuming.
d) Sensitive equipment is needed

2.4.9 Frequency Domain Decomposition

Frequency Domain Decomposition (FDD) is implemented by Brinker, Zhang and Andersen (2000), as well as Shih (1989), who used Singular Value Decomposition for each frequency line of the response spectral density matrix, known as non-parametric technique. Single values are inferred as a combination of auto-power spectra for a set of Single-Degree-Of-Freedom (SDOF) systems. It can identify the natural frequency and mode shape at a particular peak of each of the SDOF systems. The FDD technique provides the physics of the structure system by peak-picking method technique. It involves conversion of output signals into frequency response function by means of the Fourier Transform. It is a simple technique that starts by identifying the SDOF function after looking at the plot and then picks the peak of the function for each resonance on the average of the normalized singular values. The information extracted from singular values is just one single frequency line. The natural frequency corresponds to the peaks of these response plots. Mode shapes are outcome from correlation of the phase angle information with the peak magnitude (Troy M.Dye, 2002). Thus, good results of natural frequencies and mode shapes can be determined but not damping estimation.

2.4.10 Enhanced Frequency Domain Decomposition

The Enhanced Frequency Domain Decomposition (EFDD) is extension techniques of the FDD that can be used to obtain damping as well as natural frequencies and mode shapes. Dynamic characteristics of particular mode are extracted from the corresponding SDOF normalized auto-correlation functions in the time domain. FDD and EFDD is a fast, easy to use and accurate peak picking techniques in modal analysis.

2.5 ARTeMIS Extractor

ARTeMIS Extractor is software that is used to analyse ambient vibration data to determine the modal parameters which are natural frequency, mode shape and damping ratio. ARTeMIS Extractor is effective in modal identification from response only and identification of structure under real operating condition.

ARTeMIS Extractor supports three ways to create project by entering three types of information:

1. SVS Configuration File definition.
2. Universal File Format.
3. OLE Automation.

Universal File Format is used in the modal analysis that is conducted in this research study. Universal file format can be defined as data that stored in a data set which called UFF that has different numbers. There are four types of data set that are necessary to create a project in ARTeMIS Extractor which are:

1. UFF Data Set Number 15

The data sets contain the node definition which equal to the group in the SVS Configuration File format that starts with the keyword “Nodes”. It is actually the coordinates of the point that form the structures.

2. UFF Data Sets Number 82
This data set is to create the line of the geometry. They are optional data sets which contain trace line definitions and equal to the group in the SVS Configuration File format that starts with the keyword "Lines"

3. A UFF Data Set Number 2412
They are other optional data sets which have triangular surface definitions and equal to the group in the SVS Configuration File format that starts with the keyword "Surfaces". They are used to create the surface of the structures in geometry modelling.

4. UFF Data Sets Number 58
A specific node in a specific direction is contained inside these data sets and equal to the group in the SVS Configuration File format that starts with the keyword "Setup". They are the raw data file recorded by data acquisition system during experimental testing.

2.6 The Finite Element

2.6.1 Introduction to Finite Element

Finite element is a numerical technique used to obtain approximate solutions of boundary value problems in engineering. Boundary value problem is a mathematical problem in which one or more dependant variables must satisfy a differential equation everywhere within a known domain of independent variables and satisfy specific conditions on the boundary of the domain (David V.Hutton, 2004). They are used when hand calculations cannot provide accurate results.

In the finite element analysis of real structures, the actual structure is broken down into many small pieces of various types, shapes and sizes which then assembled together using the basic rules of structural mechanics, equilibrium of forces and continuity of displacements. They solve the field with discrete model. The field variables may include temperature, vibration, displacement and others.

Finite element has become a vital tool for students and engineers to solve various types of problems and unknown.

2.6.2 Basic Steps in the Finite Element Method

The finite element method (FEM) can be defined as a general numerical technique for approximating the behaviour of continua by assembly of small parts (elements). In finite element modelling, discretize the continuous structures that have infinite numbers of degree of freedoms into smaller pieces called elements for analysis is an important process. After that, stiffness is determined and assembled into the system of equilibrium equation to solve nodal displacements. Different properties and geometries will be required to cater for various types of structures and their behaviours. The procedures involved in finite element analysis consist preprocessing phase, solution phase and post processing phase (T.R. Chandrupatla & A.D. Belegundu, 1997).

2.6.2.1 Pre-processing Phase
Geometric domain of the problem is defined. The solution domain is created and discretized into finite elements by subdividing the problem into nodes and elements depending on engineering judgement. It is either small enough to give accurate result or large enough to let the computational process easier. It is assumed that a shape function depends on the physical behaviour of an element and approximation of the simulation of the actual behaviour of the problem. Equations then develop by defining the material properties, geometric properties, element connectivity as well as the boundary condition (physical constraints) and the loading. The global stiffness matrix will be constructed after assembly of the elements that represent the entire problem.

2.6.2.2 Solution Phase
A set of linear or non-linear algebraic equations will be solved simultaneously. The solution will give the nodal results for example displacement values at different nodes. Reduction of data storage requirements and computation time will be achieved by this solution technique. Gauss elimination is commonly used for static and linear problems. Additional computation and variables derivation such as reaction forces, elements stresses and strain can be done by applying the computed values before.

2.6.2.3 Post Processing Phase
The last phase is the analysis and evaluation of the solution result. Example of operations that can be accomplished using post processing software:

a) Equilibrium checking
b) Computation of element stresses
c) Deformed structural shape plotting
d) Animation of dynamic model behaviour
e) Production colour coded stress contour.
f) Calculation for factor of safety

2.7 ANSYS

2.7.1 Introduction to ANSYS

The use of finite element analysis as design tool has grown rapidly in the recent years. ANSYS has become a powerful and easy-to-use finite element program with comprehensive packages. ANSYS was released at 1971 for the first time. It contains over 100,000 lines of code and a lot of analysis can be performed through ANSYS. ANSYS has been a leading FEA program for over 20 years and now it has a completely new look and enhanced into program with multiple windows incorporating a graphical user interface (GUI) and other menus. Today, ANSYS is a vital tool in many engineering field include civil engineering. ANSYS enables engineer to perform the following task (Saeed Moaveni, 2003):

a) Construct computer models or transfer CAD models of structures, products, components or systems.

b) Study physical responses such as stress levels, temperature distributions or electromagnetic fields.

c) Apply operating loads or other design performance conditions.

d) Optimize a design early during the development process for the purpose of production costs reduction.

e) Do prototype testing in undesirable or impossible environments.

2.7.2 Overview of the ANSYS Program

The ANSYS program is divided into two levels which are the Begin level and the Processor level. Role of the Begin level acts as a gateway into and out of the ANSYS program and access certain global program controls. Database clearance and file assignment changing can be done from the Begin level. Meanwhile, most of the analysis will be done at the Processor Level which is available to accomplish a specific task in ANSYS. Figure 2.10 shows the organization of ANSYS.

Figure 2.10: Organisation of ANSYS program

There are three typical steps for analysis in ANSYS, which involves three most frequently used processors (Saeed Moaveni, 2003):

1. Pre-processing
This step is done using the PREP 7 Processor, which contains the commands needed to build a model:

a) Define element types and options
b) Define material properties
c) Define element real constants
d) Create model geometry
e) Define meshing controls
f) Mesh the object created

2. Solution
Boundary conditions and loads are applied in this step by using SOLUTION processor. Then it initiates finite element solutions.

3. Post-processing

POST1 processor is used for static or steady-state problems in this stage while POST26 processor is applied to review result over time in a transient analysis at certain point in the model. POST1 processor has the commands that allow result display and tabular listing:

a) Read results data from the results file
b) Read element results data
c) Plot results
d) List results

There are other processors such as the design optimization processor (OPT), which allows the user to perform a design optimization analysis and others that perform additional tasks.

3 Field Testing and Analysis Method

3.1 General

Two part of the analysis will be implemented in this research. First of all, the experimental modal analysis is chosen because it represents the selected bridge's experimental dynamic properties in terms of natural frequency, mode shape and damping ratio. The experimental work of ambient vibration testing will be conducted in this research. The testing is based on the master papers entitled "Modal Analysis of Three Span Bridge Using Force and Ambient Vibration Techniques" by Jeffrey D.
Hodson and "Force and Ambient Vibration Testing of Permanently Instrumented Full Scale Bridge" by Troy M. Dye.

Normally, when a building experienced dynamic vibration in its operational environment, it can be defined as ambient vibrations. For instance, sources of ambient vibrations that can affect the structures are including wind, traffic, micro-seismicity, and other known forces. Typically, ambient vibrations are eliminated or filtered out in forced vibration because they are considered to be noise in force in the signal. On the contrary, this noise is assumed to be forcing function for ambient vibration testing. The entire ambient vibration test is analysed as output-only problem since these ambient vibration are not usually measured. Ambient vibrations have been used effectively to characterise the modal properties of structure. The main limitation is that the ambient vibration might not be large enough to excite the structures or all the modes of structure.

There are two types of ambient sources excitation for these testing which are forces excite from walk movement and impact hammer. Occasionally, force excited from traffic and walk movement are used for ambient. Impact hammer is used when there is a situation that traffic does not excite sufficient vibration to the structure. Impact hammer is capable of applying peak force to structures in the form of impulse.
Consequently, an ambient source means existing sources in the surrounding area so impact on the structure is induced by hammer with unknown force of impact and random position as artificial ambient source.

In addition, real drawings for all the structures selected for the modal testing cannot be obtained so measurements of structures' dimensions were done before the testing. Some assumptions were forced to be made to carry out the analysis. Practice and trial test is carried out before the full scale testing are conducted in order to get familiar with the equipment and rewarding experience to obtain better and accurate result in the future.

A comparison is made between the two methods to see if they have comparable result. Generally, the methodology of this research is represented by the flow chart in Figure 3.1.

Figure 3.1: Flow Chart of the Research Methodology

3.2 Instrumentation Description

In order to carry out the experimental modal testing, a few prominent instruments are required and most of them are from Faculty of Mechanical Engineering Vibration Lab. There are normally three main types of measurement equipment include the excitation equipment, sensing equipment followed by data acquisition and processing equipment.

3.2.1 The Excitation Equipment

Ambient vibrations subjected to the structures usually are unknown input in operational condition. Traffic is considered as one of the ambient sources in the testing. Thus, a person with the walking or running movement on the structure is able to excite sufficient ambient vibrations to the structures and then can be applied for further analysis. This method is implemented on staircase and timber bridge testing. On the other hand, there is a case that traffic is not capable to excite the concrete bridge near the entrance of UTM. The dynamic vibration induced on the bridge is created through the use of a hammer as impact provider. The hammer is a 5803A model manufactured by Dytran Instruments Inc USA and it is shown in Figure 3.2.

Figure 3.2: Dytran Hammer used in the modal testing

3.2.2 The Sensing Equipment

In order to measure how the structures react to the dynamic vibration, several accelerometers were used. There are total numbers of four uni-axial accelerometers used in staircase testing while three accelerometers in both bridge testing. They can be categorized to two different types which three out of four are from the same manufacturer. The three accelerometers are manufactured by KISTLER each with serial number 207690, 207691 and 207692. They share the same measurement range, resonant frequency and transverse sensitivity except each has its own sensitivity.

They are used for all the testing. Meanwhile, the single accelerometer is a 3100024 model from Dytran Instruments Inc USA and it is only applied on the staircase testing. In concrete bridge testing near UTM entrance, the accelerometers are mounted on three-legged platform. All of the accelerometers are calibrated before starting any testing. Figure 3.3 illustrates the accelerometers and calibrator used in the testing. The location of the equipment along a structure is critically vital to effectively and efficiently discover possible dynamic characteristics of a structure especially mode shapes. It comes from the evidence obtained from past research done by Jeffrey Hodson on a bridge in Switzerland to include more sensing equipment than previously plan.

Figure 3.3: Accelerometers and Calibrator used in the modal testing

3.2.3 Data Acquisition and Processing Equipment

A portable system with a 16-bit analogy to digital analyser, which feed the information to a laptop for storage using data acquisition software is used to record the data measured from the modal testing. The analyser is MK II with the software of data acquisition system called PAK version 5.3 manufactured by Mueller-BBM VibroAkustic Systeme and are used at adjacent field site. The MK II analyser is shown in Figure3.4.

Figure 3.4: MK II Analyser used during the modal testing

3.3. Small Laboratory Testing

A trial test was carried out on a plate at the vibration laboratory, Faculty of Mechanical Engineering. A total of 4 sets of testing were carried out on a plate provided at vibration laboratory. These testing were focused on ambient modal testing. Sampling rate of 6000 Hz was used on all the 4 set of testing. The plate consists of nine points as illustrated in Figure 3.5. The point 1 considered as a reference point for all the 4 sets of testing. The arrangement installation of accelerometers for the 4 sets of testing is shown in Table 3.1.

Figure 3.5: Plate used in trail modal testing with points' number

Table 3.1: Location of accelerometers positioned on the steel plate

3.3.1 The Experimental Procedure

The plate was placed on a cushion so that it was in free to free condition.
There was one accelerometer acted as reference accelerometer and it put on the point throughout the whole test. Then another two accelerometers were installed on the plate according sequence in Table 3.5. The technician of vibration laboratory conducted calibration for the accelerometers before they were used. He also setup the testing equipment and made some adjustments according to the requirement until they were ready to be used.

The small impulse hammer was used to induce forces without knowing the value of the force. The forces were ignored in ambient testing. Two knocks were conducted on the plate for each set of testing. The knock was done on any location within the boundary of the plate. Then the process was repeated and the frequency was measured by accelerometers installed on the structure. The signal was processed by an MK II Analyser and the data were sent to the computer that is installed with data acquisition software named PAK version 5.3.

The software will scan the data to sort out the range of the frequency so that the data from second knock were in the range of frequency of first knock. If the second knock was out of range, the data will not be read and a warning will be shown by the software to knock on the plate again. The points of the peak accelerance inside the frequency response and the accelerance were checked on the coherence function figure to make sure the peak accelerance was within the range of 70% in coherence function shown by the software. Only the smooth frequency response and coherence function was accepted and saved for the dynamic properties analysis afterward. This test was taken to give the idea of how the modal testing is conducted and the procedures involved.

3.4 Ambient Modal Testing on a Staircase Structure

3.4.1 General Description of the Staircase

The second testing is in full scale field-testing carried out on a staircase. The staircase is selected to conduct the modal testing because the part of the landing is only supported by a beam and column, which can be categorised as hanging staircase. It is subjected to more vibration when there is loading on it. This staircase is around 4.3m tall with 28 steps and the two landings. The appearance of the staircase is shown in Figure 3.6. An AutoCAD drawing was prepared and attached in the Appendix A.

Figure 3.6: General view of the staircase

3.4.2 Staircase Testing Procedures

The first full scale ambient modal testing was conducted on the staircase in which the response of the structure was measured by ambient excitation through the movement of walking on the staircase. First of all, the instruments were setup in predetermined location. Seven sets of measurements were collected from the testing. Four accelerometers were placed on selected location for each set and the example of the accelerometers' location and the setup of instruments are shown in Figure 3.7. There was an accelerometer acted as reference, which was fixed on a location for every 7 sets. The other three accelerometers were varied from a point to another according to the set. Figure 3.8 shows the layout for the accelerometers and Table 3.2 summarises the 7 sets' details.

Figure 3.7: Accelerometer positioning and other instruments setup

Figure 3.8: Layout of accelerometers’ position on the staircase

Table 3.2: Location of accelerometers positioned on the staircase

The data was collected in different ways from how the data for plate was collected. Time history for the response recorded was used in this testing where a person would start walking or running on the staircase and induced impact on the staircase randomly without knowing the force for 2 minutes time for all the seven sets. The response was captured by the accelerometers on the predetermined location and then sent to the analyser. The response from the analyser was fed to the PAK version 5.3 Software. The procedures repeat for other six sets. The excitation from the walking or running movement on the staircase is shown in Figure 3.9 Once the testing was done, all sets of data were extracted in UFF Data Set 58 file for further analysis with ARTeMIS Extractor software to obtain the dynamic properties of the staircase.

Figure 3.9: Ambient excitation to the staircase

3.5 Ambient Modal Testing on a Timber Footbridge

3.5.1 General Description of the Timber Footbridge

The testing proceeded to another full scale ambient modal testing on the existing bridge. The existing footbridge, which located at UTM lakeside is selected to carry out field vibration data measurement. The footbridge selected for the study is located at UTM lakeside that connects the car park beside the lake to the opposite site of the lake. Figure 3.10 shows the front and side view of the bridge at UTM lakeside. The footbridge is selected for this initial research for modal testing because the testing can be conducted without interruption and the traffic on the footbridge can be controlled. Furthermore, the footbridge is located in UTM so it is more convenience to bring the testing equipments to the location. The footbridge measures 1.424 m in length and 0.185 m in width. The spans are constructed of a wooden deck supported by three main steel girders. Another 7 steel girders act as supporting beam to the span. The AutoCAD Drawing of the footbridge is attached in Appendix A.

Figure 3.10: General view of the timber footbridge at UTM lakeside

3.5.2 Timber Footbridge Testing Procedures

The second full scale testing was conducted on the timber footbridge following the measurements taken of the bridge. All the instruments were prepared before the testing including the setup and a generator for electricity. The setup of PAK version 5.3 software and MK II analyser for the response recorded was in time history. Three different period of time were conducted on the experimental testing, each with the sampling rate of 3200 Hz, 1500 Hz and 1024 Hz. The response of the bridge was saved in time block around 20 seconds, 40 seconds and 60 seconds. This presented an opportunity to compare how the resonant frequencies change given a different testing environment.

Secondly, three accelerometers were used for the testing and the location to position strategically the accelerometers were determined. There were total 8 sets of measurements which every set shared the same location of one reference accelerometer and two different locations for another two accelerometers. Figure 3.11 shows the layout of the accelerometers position while Table 3.3 systematically shows the arrangement of accelerometers for the 8 sets of measurements. The accelerometers were placed on the location where support of the hand reel laid. This approach was chosen because it could conveniently recognize the position and its appropriateness to place the accelerometers. The example of location is shown in Figure 3.12

Once the setup was completed including placing the accelerometers, the footbridge was subjected to the ambient vibration by walking and running movement of ambler, which was assumed as ambient source of traffic. Figure 3.13 clearly shows the movement that excite vibration to the timber footbridge. The response from the excitation started to be captured by the accelerometers and sent to the MK II analyser, lastly fed to the laptop that installed with PAK version 5.3. The process was repeated for the other 7 sets for three time settings.

After series of ambient measurement, all data were saved and retrieved from the laptop in UFF Data Set 58 file and collected and would be analyzed by ARTeMIS Extractor to obtain the dynamic properties of the timber footbridge.

Figure 3.11: Layout of accelerometers' position on the timber footbridge

Table 3.3: Location of accelerometers installed on the timber footbridge

Figure 3.12: Accelerometer positioning on the timber footbridge

Figure 3.13: Ambient Excitation to the timber footbridge

3.6 Ambient Modal Testing on Concrete Bridge

3.6.1 General Description of the Concrete Bridge

The third dynamic testing takes place on a concrete bridge. The main reason to conduct the test on this selected bridge is because its location is easily accessible, which is therefore convenient to transport all the testing equipments. Another reason is that the traffic can be controlled to prevent interruption during the testing. Application was made to Security Director and permission was granted to conduct the dynamic testing. UTM Traffic Officers had been helpful and effectively control the traffic during the testing.

The bridge which has four lanes consisted of thirty nine pre-cast beam with the depth of 0.528 m and width 0.5 m. The bridge is 10.8 m long and 21.5 m wide with two walkways on both sides. There is a 5.8 m wide tuft and plants in the middle of the bridge. Besides the “plants” zone, there are two runways each with 6.1 m wide. A picture of the bridge is shown in Figure 3.14. The AutoCAD drawing attached in Appendix A shows the layout plan of the bridge.

Figure 3.14: General view of the concrete bridge

3.6.2 Concrete Bridge Testing Procedures

The concrete bridge was tested using a hammer as the vibration source. The accelerometers were position strategically at pre-determined location as illustrated in Table 3.4 and Figure 3.15. There were in total twelve sets of data collected from the testing. Instruments were made ready and setup was properly prepared before the test including accelerometers calibration, setup of data acquisition software and a set of generator for electricity. The bridge was excited with vibration from hammer continuously for around 3 minutes or sampling rates 375 Hz. The accelerometers were mounted on three-legged platform then position at the pre-determined location.

One accelerometer acted as reference and another two accelerometers were shifted according to the pre-determined set of data. Figure 3.16 gives the example of how the accelerometers position on certain location. Hammer was used because traffic did not excite the bridge after some try. The hammer forces not only will not be measured and remain unknown, the hammer also will randomly impact bridge on random location just as shown in Figure 3.17. The procedures were repeated for all the data set

Figure 3.15: Layout of accelerometers' position on the concrete bridge

Table 3.4: Location of accelerometers installed on the concrete bridge

Figure 3.16: Accelerometer mounted on three legged platform

Figure 3.17: Ambient excitation to the concrete bridge

Once the data were collected, they were recorded into the laptop and then extracted in the form of UFF Data Set 58 file which were required in ARTeMIS Extractor, ambient vibration software for dynamic characteristic identification.

3.7 Analysis using ARTeMIS Extractor

The ambient vibration data consisted of time histories for every accelerometer. Figure 3.18 gives the illustration of typical time history of accelerometer. The complete set of time histories for Timber Bridge and concrete bridge are in Appendix B. The raw data is too long to be shown here but an example from part of the data is attached in Appendix B. All the field data were transferred in UFF Data Set 58 file from the data acquisition system for analysis.

Figure 3.18: Typical accelerometer time history

There have been myriad of different analysis procedures to extract the modal characteristic of structure. However for this research, data processing of the data is performed through the use of ARTeMIS Extractor programme developed by Structural Vibration Solution APS. The Enchance Frequency Decomposition Peak Picking Technique is employed in the analysis to extract the dynamic characteristics from the selected structures.

UFF Data Set number 15, 82 and 2412 are essential to create the geometry of the structures in ARTeMIS Extractor. They are together in a single text file. The full set of text file contained of geometry of all the structures in testing are attached in Appendix B. The UFF data sets number 58 containing the measured data are located separately in different text files. The number of text files depends on the data set determined for each structure.

The Analysis begins with choosing Universal File Format to create the new project in the new project dialog. After that, the data sets in a Universal File Format Data Set Number 15, 82 and 2412 in a single text will be loaded. It will automatically show how many nodes, trace lines and surfaces when the file is successfully loaded. It was then followed by loading the file containing the measured data. The measured time series need to be stored in separate file for each data set. The structure was loaded in "Project Control" with the information and geometry. Figure 3.19 shows the layout to create new project.

Next, the "Signal Processing Configuration" wizard is performed on the beginning of the analysis in defaults setting. The next step can either be testing the "Trial Configuration" or straight away to "Process Data". In "Process Data" wizard, only the spectral density matrices estimation is selected to be executed. The steps are illustrated in Figure 3.20 and Figure 3.21. After the data processing, "Enhanced FDD Peak Picking" mode will be used to extract the dynamic characteristics. The peak of the graph is chosen and it represents one mode shape with natural frequency and damping ratio. The animation of the deformed geometry can be viewed. The number of mode can be extracted from a structure will depend on the data set collected from the experimental testing.

The following Figure 3.22 and 3.23 show how to peak picking, the example of animation of deformed structure and geometry of all the selected structures done for this research. The results of the analysis are discussed further in analysis chapter.

Figure 3.19: Layout window of creating new project in ARTeMIS Extractor

Figure 3.20: Layout window of the signal processing configuration

Figure 3.21: Layout window of data processing

Figure 3.22: Layout window of EFDD Peak Picking

Figure 3.23: Layout window of deformed structure animation

4 Finite Element Modelling

4.1 General

A theoretical testing is conducted base on the finite element method to perform the analysis in order to obtain the dynamic properties of the selected structure. It will produce the theoretical frequency and mode shape. ANSYS is chosen as the finite element software to be used to model the selected structures. Finite element modelling is performed with the dimensions and elevation obtained from measurements.

However, finite element analysis is just an approximation method of analysis to obtain the dynamic properties. The accuracy of the result obtained from the finite element modelling is quite doubtful and may not be perfectly accurate. Therefore, the experimental modal testing as discussed in the last chapter, the approach that holds the most promise is applied to obtained reliable results.

4.2 Finite Element Modelling of the Staircase

An actual drawing of the staircase was unable to retrieve so the modelling was started with the measurement of dimension and elevation of the staircase. A finite element modelling had done using ANSYS according to the measurement taken. The modelling was done in text file with ANSYS commands then the text file was read as input from ANSYS. All commands were done in text file format because it is convenient to modify, load and perform analysis. The full command to model the staircase is attached in Appendix C.

Some assumptions were made on this staircase which were:

1. The concrete strength for the staircase is 30 N/mm2 and it gives the elastic modulus of 24.5 x 109 N/m2. Poisson ratio and density are 0.2, 2400 N/m2. The whole components of structure share the same material properties.

2. The weight of steel hand reel was negligible and was not included in the modelling.

3. The element of staircase was assigned as solid or SOLID92 in ANSYS. It is a 3-D 10-Node Tetrahedral Structural Solid. SOLID92 in ANSYS has a quadratic displacement behaviour and is well suited to model irregular meshes. The element is defined by ten nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element also has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities.

4. As for the boundary condition, the staircase was restraint in all direction at 3 area, surface below the first step, below surface of column and the surface of the last step. Figure in Appendix C shows the surfaces that restraint.

5. Free meshing was used with “smartsize” of 5 that represent coarseness of the meshing in ANSYS. Smaller size will produce finer meshing.

6. Modal Analysis in ANSYS was done with subspace extraction mode and 20 modes were extracted from the analysis.

Figure 4.1 shows the interface of ANYS with the staircase model. The results of from the analysis of staircase using ANSYS were extracted and will be explained further in analysis part.

Figure 4.1: Interface of ANSYS with the staircase model

4.3 Finite Element Modelling of the Timber Footbridge

Since the actual drawing of the footbridge is also not available so the modelling will be started with the measurement of the footbridge dimension. The measurement was done using measurement tape. All the components of the footbridge were measured repeatedly then an average data ware used to prepare an AutoCAD drawing as attached in Appendix A. The measurements of the footbridge were also applied for the finite element modelling. The following assumptions were made:

1. The beams of the footbridge are steel girder and the size of RHS 150x85x10 is used as main beam and RHS 150x75x10 used as supporting beam. The timber planks make of “cengal” type acted as a slab. The material properties of steel girder are elastic modulus of 200 x 109 N/m2, density 7850 N/m3 and Poisson ratio 0.3. The timber will have elastic modulus of 19 x 109 N/m2, density of 980 N/m3 and Poisson ratio 0.2.

2. The weight of hand reel was negligible so it was not included in the modelling.

3. The element of the footbridge consists of beam as the steel girder and shell as the timber plank. BEAM4 was assigned for the beam element in ANSYS while SHELL63 was assigned for the shell element. BEAM4 is 3-D uni-axial elastic beam with tension, compression, torsion, and bending capabilities. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. SHELL63 is an elastic shell that has both bending and membrane capabilities. Both inplane and normal loads are permitted. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z-axes.

4. As for the boundary condition, the footbridge was restraint at the main beam with four key points in ANSYS in all direction as shown in Appendix C.

5. Mapped meshing was used with line mesh on the structure modelling in ANSYS.

6. Modal Analysis in ANSYS was done with subspace extraction mode and 20 modes were extracted from the analysis.

Figure 4.2 shows the interface of ANSYS with the timber footbridge modelling. The results from the analysis of footbridge using ANSYS were extracted and will be explained further in analysis part.

Figure 4.2: Interface of ANSYS with the timber footbridge model

4.4 Finite Element Modelling of the Concrete Bridge

The measurement of dimension and elevation of the concrete bridge was carried out due to unavailability of the drawing of bridge. There were difficulties to obtain the measurement of the bridge using measurement tape due to the traffic problem. Surveying method was chosen in order to get the measurement. The problem was consulted to Encik Radzuan in Surveying Unit to seek from his advice. Recommendation and advice from Encik Radzuan was to use "inverted rod method" with Sokkia brand level and levelling rod. The measurement took part in two days with first day to observe the condition of the bridge to suggest suitable place to obtain the measurement. The surveying work took part on the second day by using inverted rod method. Each dimension was taken three times at different location. Then the average values were calculated and used as final result to prepare an AutoCAD drawing was prepared and attached in Appendix A.

After that, a finite element modelling was done using ANSYS. Same as before, the modelling was done in text file with ANSYS commands then the text file was read as input from ANSYS. The full command to model the bridge is attached in Appendix C.

Some assumptions made on this bridge were:

1. There are two parts of components with different material properties. The concrete strength for the bridge is 45 N/m2 for the pre-cast beam and it gives the elastic modulus of 30.1 x 109 N/m2. The deck is assumed to have concrete strength of 30 N/m2 and the elastic modulus of 24.5 x 109 N/m2. Poisson ratio for both are 0.2 and density of 2400 N/m3

2. The weights of shoulder and hand reel were negligible and will not be considered in the modelling.

3. The element of bridge was assigned as solid or SOLID185 in ANSYS. It is a 3-D 8-Node orthotropic Structural Solid. SOLID185 in ANSYS has a quadratic displacement behaviour and is well suited to model irregular meshes. The element is defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, and z directions. The element also has plasticity, hyperelasticity, creep, stiffening, large deflection and large strain capabilities. Mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials is available on this element.

4. As for the boundary condition, the bridge was restraint in all direction at surface cross section area of beams and deck. Figure in Appendix C shows the surfaces that restraint.

5. Mapped mesh was used for this bridge modelling. A mapped area mesh contains only quadrilateral elements. The structure was defined into element division in order to carry out the mapped mesh.

6. Modal Analysis in ANSYS was done with subspace extraction mode and 20 modes were extracted from the analysis.

Figure 4.3 shows the interface of ANYS with the concrete bridge model. The results from the analysis of concrete bridge using ANSYS were obtained.

Figure 4.3: Interface of ANSYS with the concrete bridge model

5 Analysis of Results

5.1 Introduction

The results to be discussed in this chapter will include the experimental analysis and finite element analysis. All the field data from selected structures were analysed in ARTeMIS Extractor. At the same time, finite element analysis was analysed in ANSYS to obtain the dynamic characteristics of the structures. Lastly, comparison between the dynamic characteristics from experimental analysis and finite element analysis were made.

5.2 Ambient Vibration Data

The method to analyse the ambient vibration data in ARTeMIS Extractor is known as Enhanced Frequency Domain Decomposition Peak Picking technique since ambient vibration is an output-only problem. The first step in the method is to perform the FDD Peak Picking, and the second step is to use the FDD identified mode shapes to identify the Single-Degree-Of-Freedom (SDOF) Spectral Bell functions and from these SDOF Spectral Bells estimate all modal parameters.

There are a few main steps involve in the EFDD technique. Spectral density matrices are estimated from the raw time series data and the singular value decomposition of the spectral density matrices will be performed. Average of the singular value of all multiple data sets is performed followed by the second set if there are multiple data sets. All these steps are automatically done when the data is processed using “Process Data” wizard. Next peak picking of the average singular values (SV) is performed using Enhanced FDD Peak Picking editor. The modes are estimated purely by signal processing so the technique is a completely nonparametric technique.

5.2.1 Dynamic Characteristics of Tested Staircase

Determination of natural frequencies was done through the singular value plot created by EFDD algorithm. Figure 5.1 illustrates the SV plot of ambient data file. The peak of the SV indicated either a natural frequency or noise. After the peak picking on the SV plot, four modes were found in the frequency range of 16 to 254 Hz. Damping ratios were also obtained from each mode. One drawback to the number of natural frequency was that all accelerometers used in the testing were uniaxial so only four natural frequencies can be made. Table 5.1 summarizes the natural frequency and damping ratio for each mode using EFDD method.

Peak amplitude and relative phase value that were picked from the respective response plot will produce the mode shapes of the structure. It should be pointed out that the most of the mode shapes were in vertical mode since the uni-axial accelerometers were used. There are no transverse instruments to record the vibration so the mode shapes are limited. Another drawback was that the mode shape for the steps of staircase was modelled as line due to the position of accelerometers.

Therefore, a strong relationship can be seen between pre-determined position of accelerometers and mode shapes. Three dimensional representation of the mode shape can be seen in Figure 5.2

Figure 5.1: SV plot of the staircase from ARTeMIS Extractor

Table 5.1: Experimental natural frequencies and damping ratios of the staircase

Figure 5.2: Experimental mode shapes of the staircase

5.2.2 Dynamic Characteristics of Tested Timber Footbridge

There were a total of 3 different sampling rates of data collected for this structure. After the "Data Process", the three analysis displayed different SV plot of data and they are illustrated in Figure 5.3 to Figure 5.5. However, the sampling rates setup with longer period of vibration excitation produced clearer peak than the rest. It is reviewed that longer period of excitation recorded, the better and more modes can be gained. The data sets of 60 seconds were used for EFDD peak picking. The natural frequency range for the structure was found to be from 4 to 144 Hz. The damping ratios range from 3.5 to 11.2 %. The natural frequencies and damping ratios obtained from EFDD method are shown in Table 5.2

The mode shapes of the system were extracted using EFDD method as had been mentioned previously. It is observed that the mode shapes were successfully produced, however for only eleven modes. The mode shapes of the bridge that were produced by the ambient vibration were very clean and easily distinguished between each other. Figure 5.6 shows all the mode shapes of the timber bridge.

Figure 5.3: SV plot of the timber footbridge for 60 seconds data set

Figure 5.4: SV plot of the timber footbridge for 40 seconds data set

Figure 5.5: SV plot of the timber footbridge for 20 seconds data set

Table 5.2: Experimental natural frequencies and damping ratios of the timber footbridge

Figure 5.6: Experimental mode shapes of the timber footbridge

5.2.3 Dynamic Characteristics of Tested Concrete Bridge

By inserting all data sets of the tested concrete bridge, a natural frequency measurement was easier to be documented. The SV plot was cleaner and had less noise, which showed clearer peak because hammer was used to excite the structure comparing to the plot in staircase analysis which took ambient traffic as excitation. Figure 5.7 shows the SV plot of ambient data of concrete bridge.

There were twelve modes obtained and the natural frequency ranged from 16.1 to 98.5 Hz. Damping ratios found for the twelve modes were from 1.69 to 11.79 %. Complete sets of natural frequency and damping ratio of bridge can be seen Table 5.3. Twelve mode shapes were produced from EFDD peak picking technique along their natural frequency. It can be pointed out that the mode shapes of this bridge were in vertical mode. There might be other modes that exist but unable to obtain because the accelerometers used for the testing were uni-axial. They only measured the vibration in single direction. Figure 5.8 shows the three-dimensional representation of the mode shapes.

Figure 5.7: SV plot of the concrete bridge

Table 5.3: Experimental natural frequencies and damping ratios of the concrete bridge

Figure 5.8: Experimental mode shapes of the concrete bridge

5.3 Finite Element Analysis using ANSYS

In order to compare the result of experimental modal testing, finite element analysis was performed for all the structures. A typical natural frequencies and mode shapes for the structures were obtained from the analysis. Finite element analysis for this research did not produce damping ratio. The maximum displacement (DMX) is important value for structural health monitoring and future generic design. The natural frequencies of the three structures are summarized from Table 4.4 to 4.6.

Meanwhile, the three-dimensional mode shapes are illustrated in Figure 4.9 to 4.11.

Table 5.4: Theoretical natural frequencies and dominant direction of the staircase

Figure 5.9: Theoretical mode shapes of the staircase

Figure 5.9: (continued) Theoretical mode shapes of the staircase

Table 5.5: Theoretical natural frequencies of the timber footbridge

Figure 5.10: Theoretical mode shapes of the timber footbridge

Figure 5.10: (continued) Theoretical mode shapes of the timber footbridge

Table 5.6: Theoretical natural frequencies of the concrete bridge

Figure 5.11: Theoretical mode shapes of the concrete bridge

Figure 5.11: (continued) Theoretical mode shapes of the concrete bridge

5.4 Comparison of Experimental Result and Finite Element Analysis

The use of finite element analysis created an opportunity to compare the reliability of the data. A side-by-side comparison can be made for staircase, timber footbridge and concrete bridge and there were similarity and diversion between each other. It is obvious that there were more modes created from finite element analysis than the experimental analysis.

In the case of staircase, there were two modes that were almost alike in experimental and theoretical results. Natural frequency from mode 2 in the ambient vibration testing which value was 40.19 Hz was almost similar with the natural frequency from mode 4 with 43.676 Hz that was obtained from the finite element analysis. Another similar mode was mode 3, results from ambient vibration testing was more or less similar to the mode 10 from the finite element analysis. Each of them produced natural frequency of 115.4 Hz and 112.07 Hz where the values were quite similar. Figure 5.12 shows the comparison of the mode shapes of the staircase.
Table 5.7 provides the comparison of experimental frequencies and finite element.
Experimental modal analysis Finite element analysis

Figure 5.12: Mode shapes of the staircase comparison between experimental modal analysis and finite element analysis

Table 5.7: Frequency comparisons between the experimental analysis and FE analysis of the staircase

The mode shapes of the timber footbridge obtained from ambient vibration testing and finite element analysis showed high degree of similarity. However, there were discrepancies in natural frequencies for the two methods. Only seven modes showed some similarity and they are illustrated in Figure 5.13. The comparison of frequencies of the seven modes are summarised in Table 5.8

Figure 5.13: Mode shapes of the timber footbridge comparison between experimental modal analysis and finite element analysis

Table 5.8: Frequency comparisons from experimental modal analysis and FE analysis of the timber footbridge

The concrete bridge had the same result of comparison. Both the experimental and theoretical analysis had produced several similar mode shapes but with different natural frequency in each mode. It was observed that seven mode shapes were found slightly similar. In contrast, the natural frequencies for the similar modes from ambient vibration testing were in the range of 20 Hz less than the natural frequencies in the finite element analysis. Comparison of frequencies from seven similar mode shape of concrete bridge is shown in Table 5.9.
Side by side comparison of the mode shapes can be found in Figure 5.14.

Table 5.9: Frequency comparisons from experimental modal analysis and FE analysis of the concrete bridge

Figure 5.14: Mode shapes of the concrete bridge comparison between experimental modal analysis and finite element analysis

Comparison of the result themselves can be very beneficial. In comparing the two methods, the following criteria were considered: required instruments and time consumption, accuracy and subjectivity.

5.4.1 Required Instruments and Time Consumption

Both methods require analysis software to perform modal analysis in order to obtain the dynamic characteristics of the structures. Nonetheless, other instruments as mentioned in the previous chapters are required to conduct the experimental modal testing. The configuration of the instruments for experimental modal testing is very imperative to extract good quality data. The configuration of the instruments is time consuming because improper configuration will give erroneous data. There was a case during the testing of the timber bridge where the data collected from the wrong configuration could not be analysed in ARTeMIS Extractor and the testing was repeated.

The time necessary to implement these methods vary depending on the situation occurred. The ambient vibration analysis takes a longer duration to be completed. However, it is essential to have longer period of time to excite and measure ambient vibration due to the fact that ambient analysis exploited method that was entirely foreign at the onset of the research and much more trial and error was involved.

There was another situation that the longer period of time to record the data, the better end result it will be as it can be seen from the timber bridge testing. The testing will consume more time if the structures are larger in size. The accelerometers are limited so there are more sets of data need to be collected.

Meanwhile, theoretical modal testing only requires time to programme the command to perform analysis in ANSYS to obtain the dynamic characteristics of the structure.
It is time consuming in the case when the structure is complicated to model.

5.4.2 Accuracy and Subjectivity

In the comparison part, there are a few modes from the experimental modal analysis, which were slightly similar to the finite element analysis. Figure 5.12 to Figure 5.14 show the few examples of similarity of both results. There are several reasons that might be suggested to explain some of the discrepancies in the natural frequencies and also the mode shapes. Firstly, different assumptions were made for both analyses in order to obtain the dynamic characteristics. There was no actual drawing retrieved for the selected structures so assumptions were made in the finite element analysis. The results of the analysis varied from each other due to that.

It should be noted that the task of picking information off the singular value plots is at times very subjective in experimental modal analysis. There are difficulties to determine where the actual peak occurs in the plot if the noisy data is used. The subjectivity nature of the characterization process that causes some errors in peak picking is inevitable so there are few satisfactory modes produced in the end.

Furthermore, the types of accelerometers used in experimental testing are only able to measure vibration in one direction. All the uni-axial accelerometers are positioned in vertical direction so the modes produced from experimental analysis mostly are deformed vertically, torsionally or both. There is no lateral deformation in experimental results. However, finite element analysis produced all the deformation of structures include lateral mode. Due to that reason, fewer modes are able to be extracted from experimental analysis while the finite element analysis considers deformation in all directions.

6 Conclusions

6.1 General Conclusions

The significance of modal analysis through modal testing on few structures created opportunity to compare the accuracy of dynamic measurements. The repeatability of finding dynamic characteristics had enhanced the performance of modal analysis and provided a high level confidence in instrument measurements and analysis procedure.
Based on the overall studies on theoretical and experimental modal analysis of three selected structures, conclusions can be drawn as follows:

1. Instrumentation work and measurement of ambient vibration test have been carried out on three selected structures: a staircase, a small timber footbridge and a short span concrete bridge.

2. The experimental dynamic properties of the three selected structures have been successfully obtained from the ambient modal analysis compared with the finite element analysis.

3. It is observed that there are two modes in experimental testing on the staircase structure which exhibit similar natural frequencies and mode shapes with those obtained from finite element analysis.

4. Similar good comparison is obtained for the timber footbridge and concrete bridge structures as both have seven similar mode shapes with those in the finite element analysis.

5. Ambient vibration testing can be conveniently applied for long term health monitoring of structures due to its advantage of not causing service interruption to the test structures.

The problem of different modes between theoretical and experimental results will be brought forward to the next stage for further study and analysis. To clarify, modal updating will be carried out in the finite element analysis by referring to the experimental testing results. Model updating will involve changes in modulus of elasticity and comprise the value of damping ratio into analysis. This is critically imperative considering long-term monitoring of the structures using ambient vibration method.

6.2 Recommendations

To improve the efficiency of future ambient vibration data analysis, it would be necessary to have a longer period of time for data collection. It will not only provide valid data to be analysed but also produce clearer peak in singular value plot to ease the process for EFDD peak picking to obtain the modal parameters. Another option is to utilize more accelerometers on testing which can reduce the number of data sets needed to be collected for a structure.

It is also recommended that the suitable type of accelerometer to be deployed in the ambient modal testing for example tri-axial accelerometer. Tri-axial accelerometer can measure the vibration in three direction rather than uni-axial accelerometer. If this option is unavailable, another suggestion is to mount some of the uni-axial accelerometers on different axis from each other. It could greatly increase the number of modal parameter estimation for certain structure if the procedures are implemented.

Another recommendation is to choose an appropriate structure for testing in the condition that the actual drawing of the structure can be obtained. It is essential for finite element modelling to have proper dimension and elevation. Valid material properties of structural components such as modulus of elasticity, Poisson ratio are important to perform the modal analysis. Accurate dimensional measurement and valid material properties would give more accurate results of the finite element analysis.

A repetitious monitoring system is necessary in order to study the effects of ambient vibration on the structure and other unknown sources on the structure. Finally, the recommendations would be able to enhance the performance of future ambient vibration analysis whereby more refined dynamics characteristics could be obtained from the testing.

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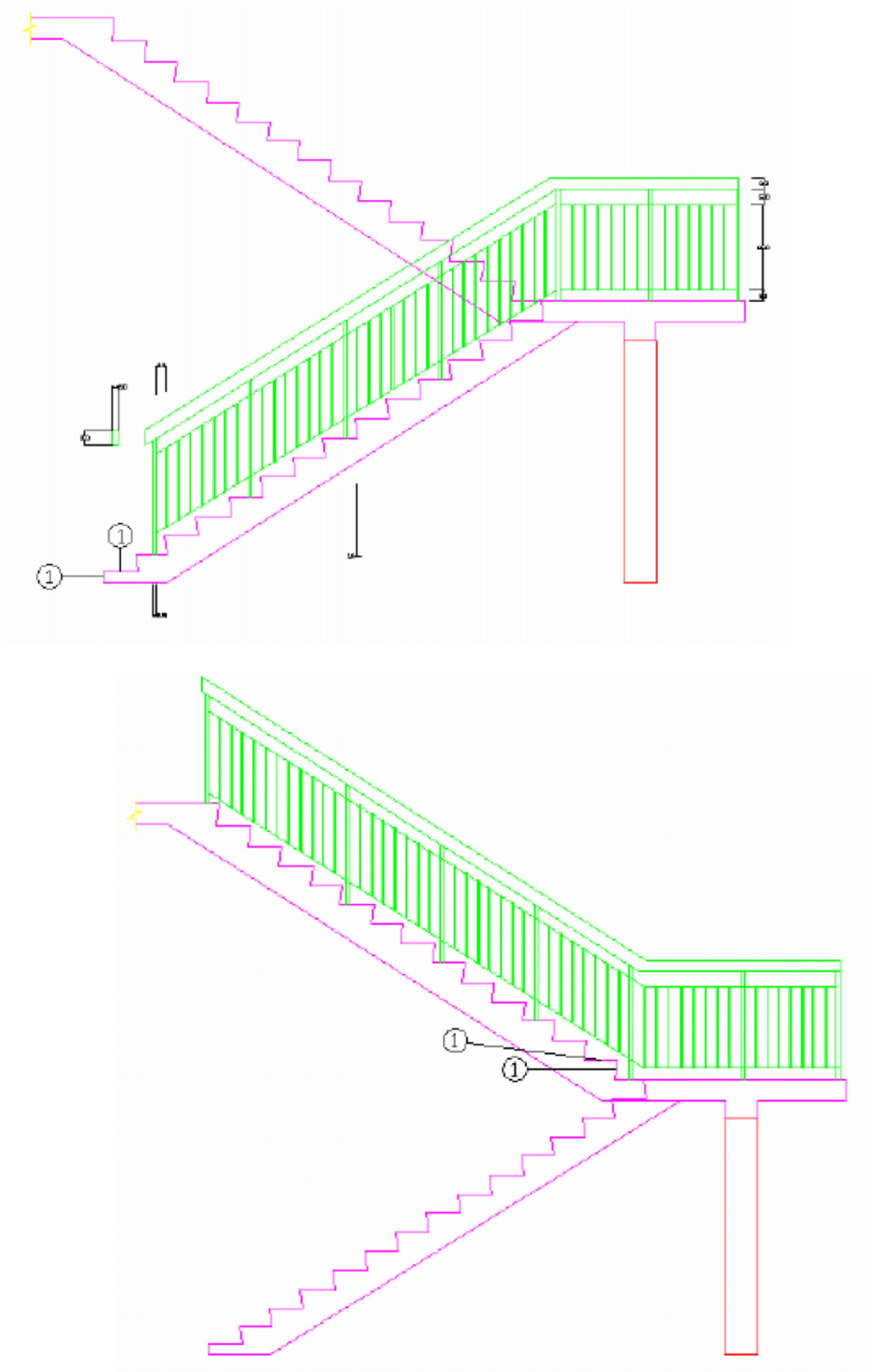
Appendix A

The tables show the sizes for the rise and thread starting from ground of the staircase which is located opposite vibration laboratory, Faculty of Mechanical Engineering.

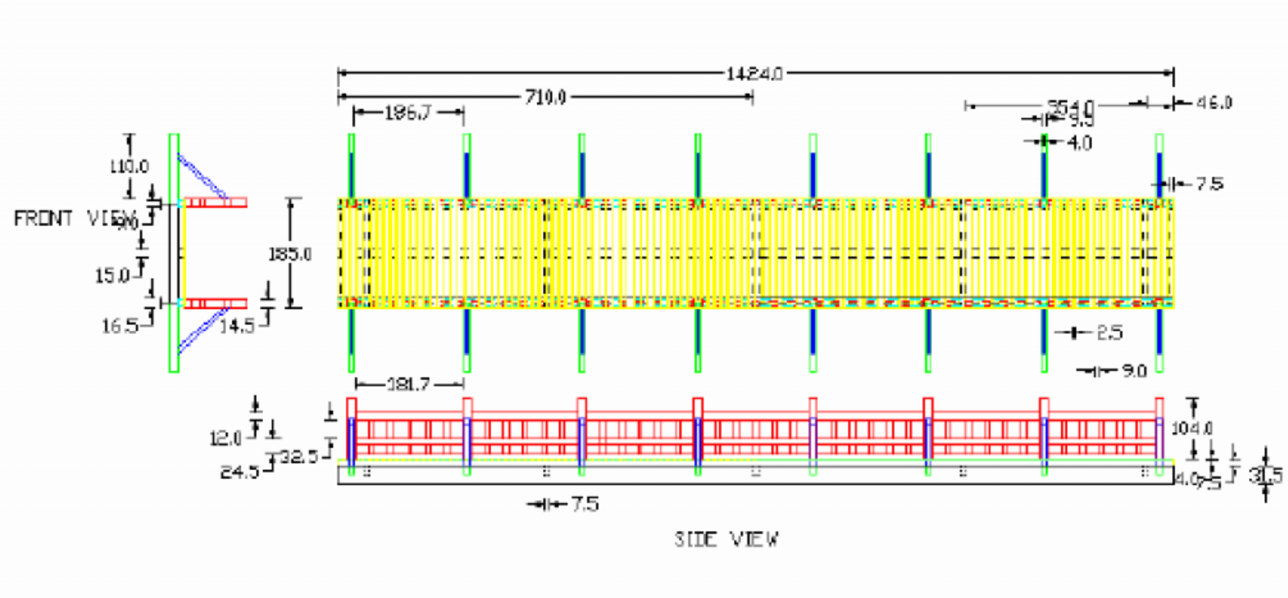
Table A.1: Sizes of rise and thread

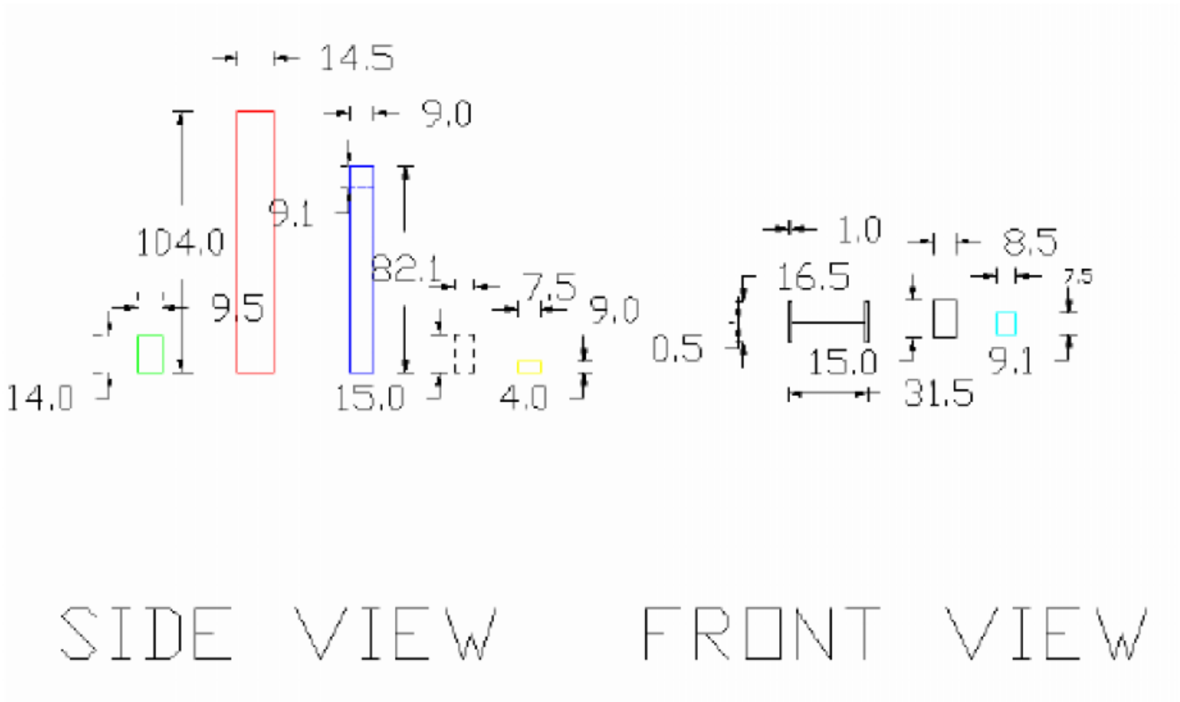
Table A.2: Sizes of rise and thread



Side View of the Staircase Structure

Layout Plan of the Staircase Structure





Layout Plan of the Timber Bridge Structure

Layout Plan of the Concrete Bridge Structure

Appendix B
Timber footbridge near UTM lake

Figure B.1: Time history plots set 1 for the timber footbridge

Figure B.2: Time history plots set 2 for the timber footbridge

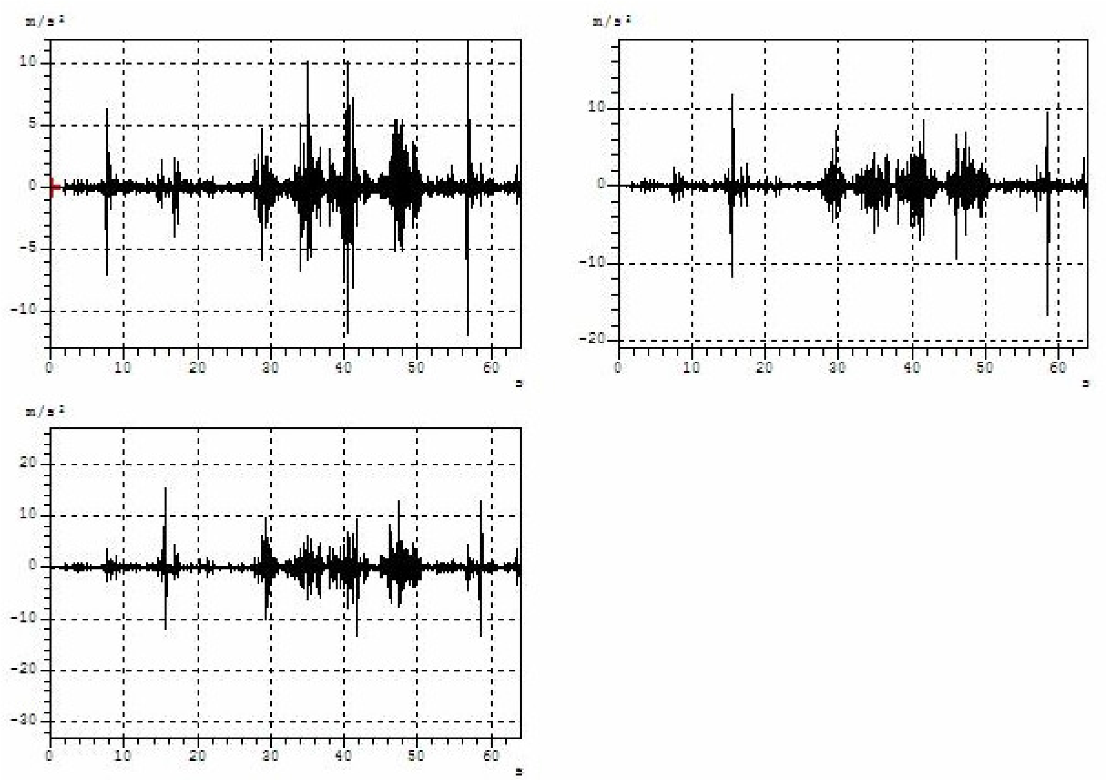


Figure B.3: Time history plots set 3 for the timber footbridge

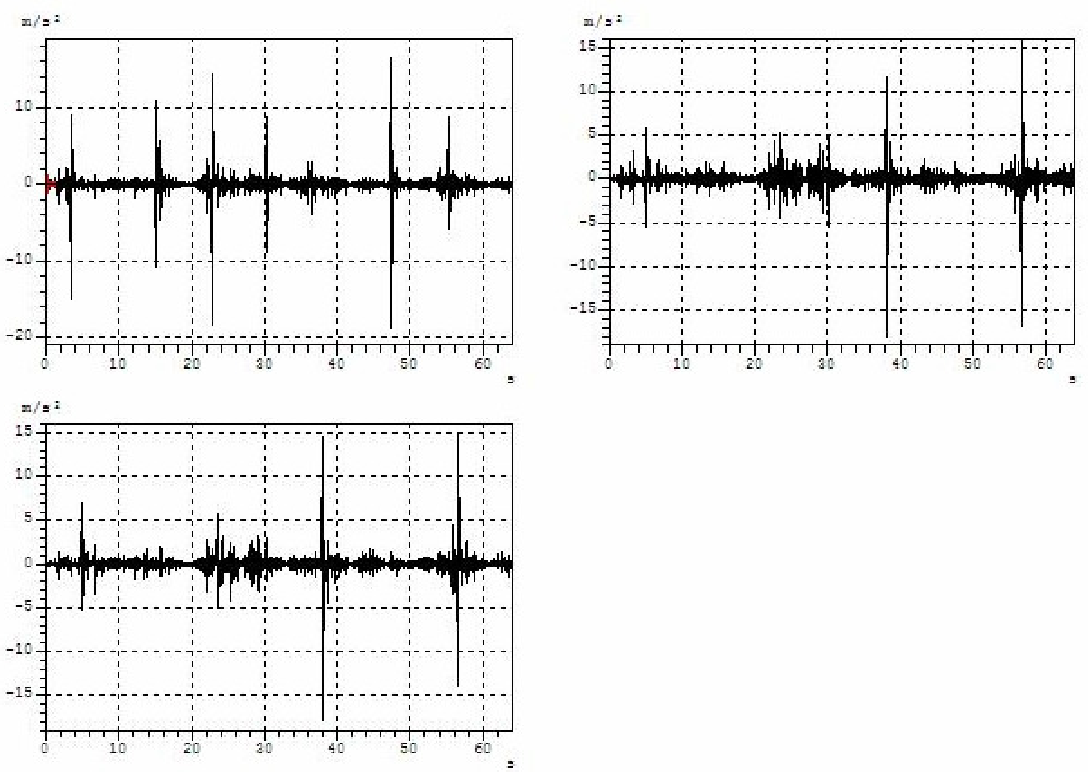


Figure B.4: Time history plots set 4 for the timber footbridge

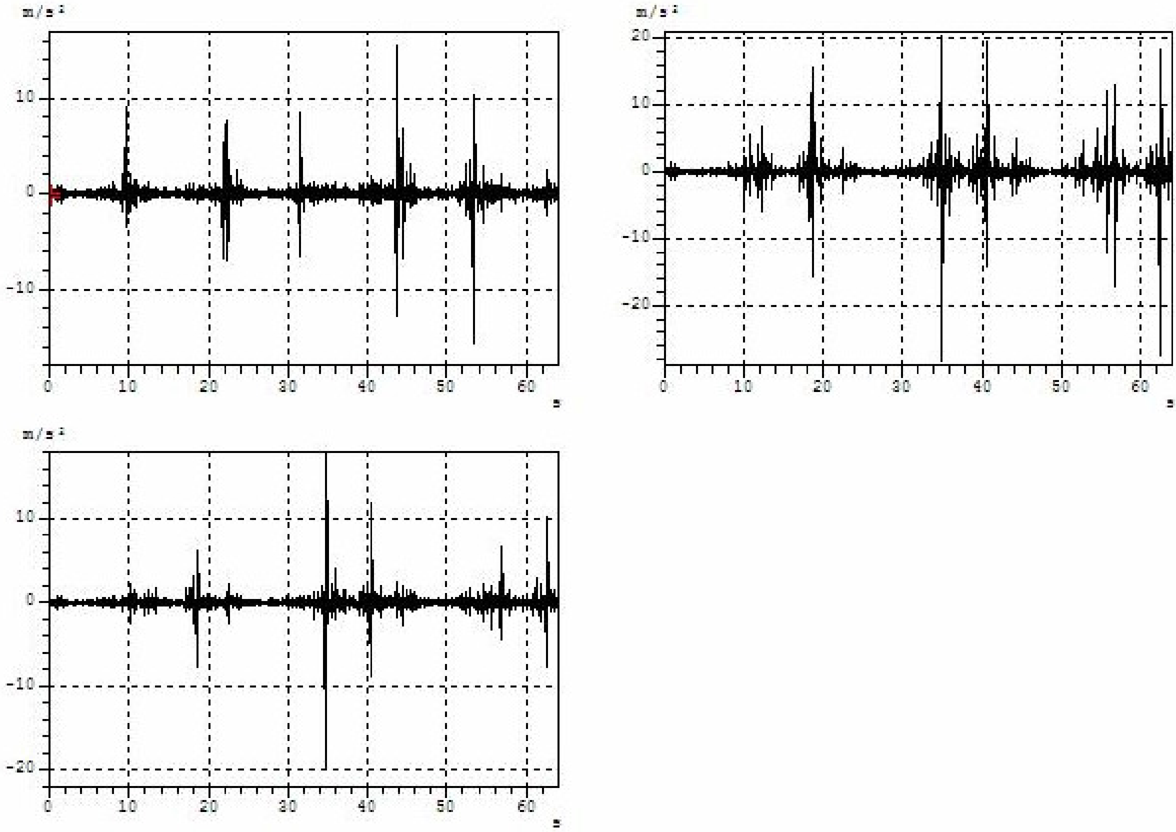


Figure B.5: Time history plots set 5 for the timber footbridge

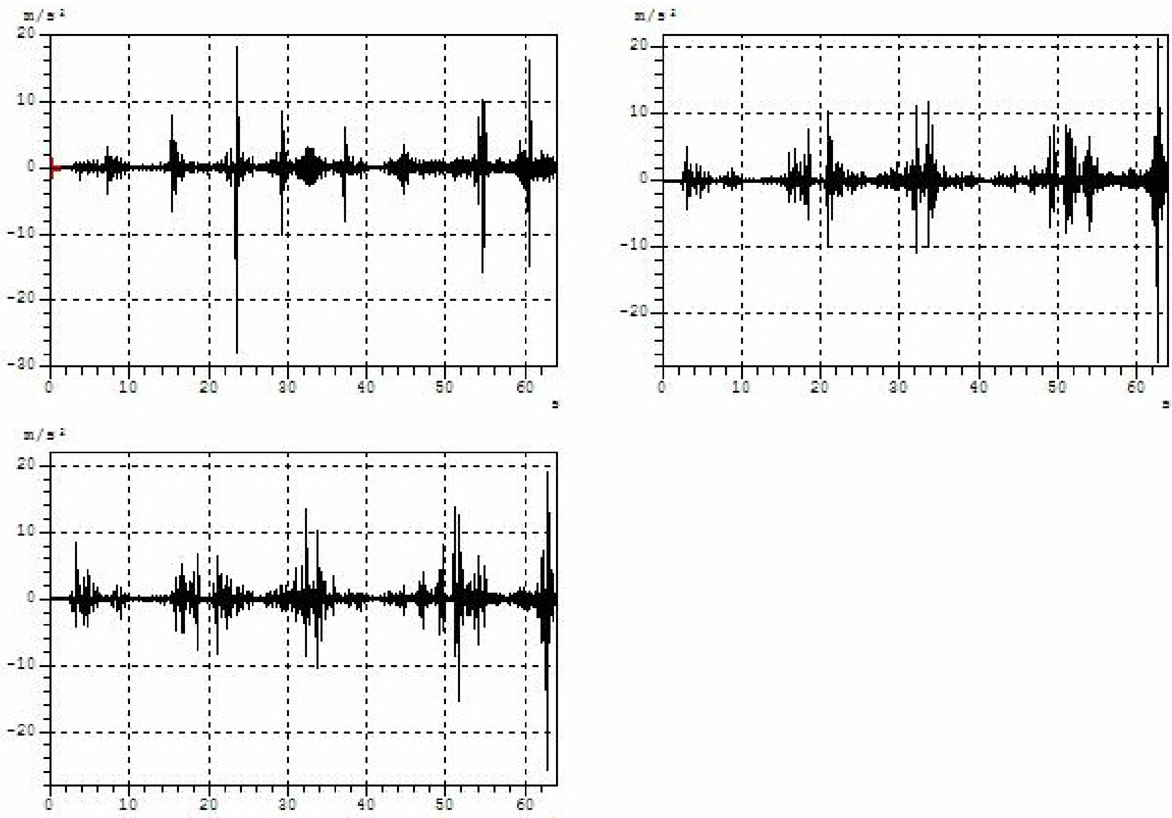


Figure B.6: Time history plots set 6 for the timber footbridge

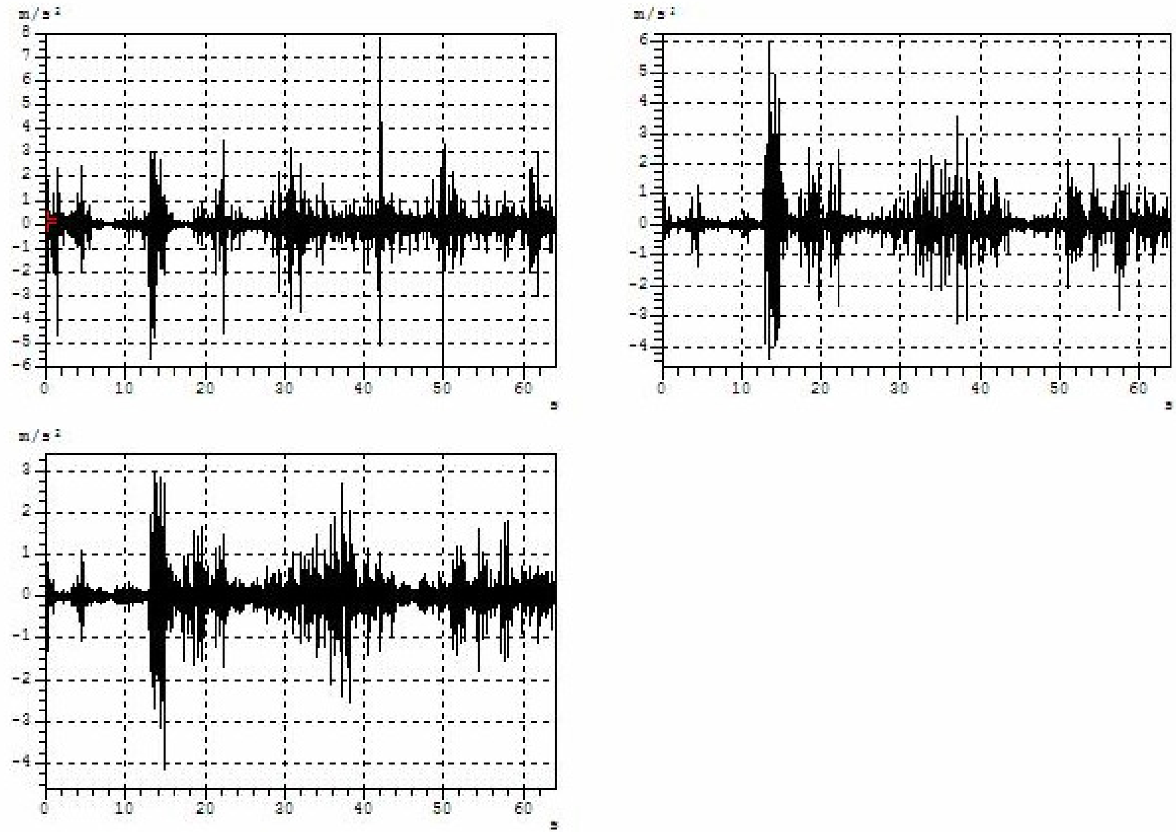


Figure B.7: Time history plots set 7 for the timber footbridge

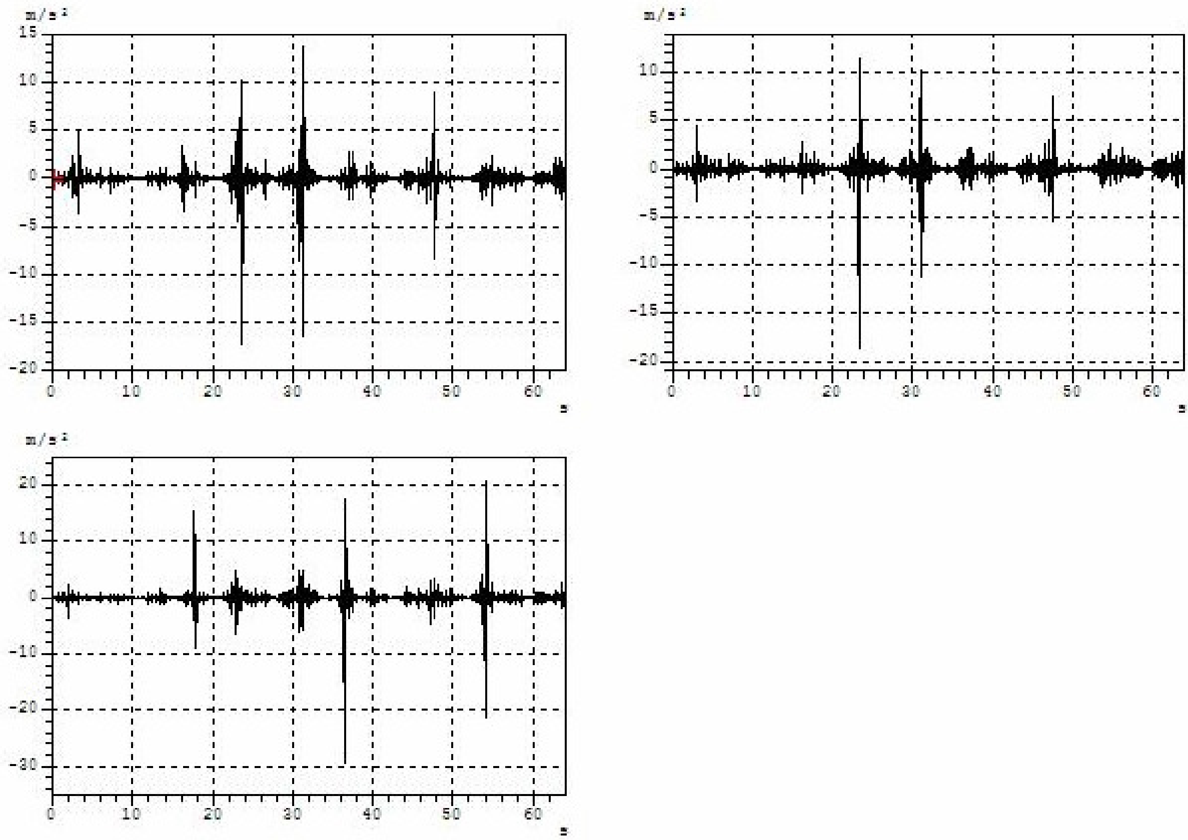


Figure B.8: Time history plots set 8 for the timber footbridge

Time History Plots for Concrete Bridge

Figure B.9: Time history plots set 1 for the concrete footbridge

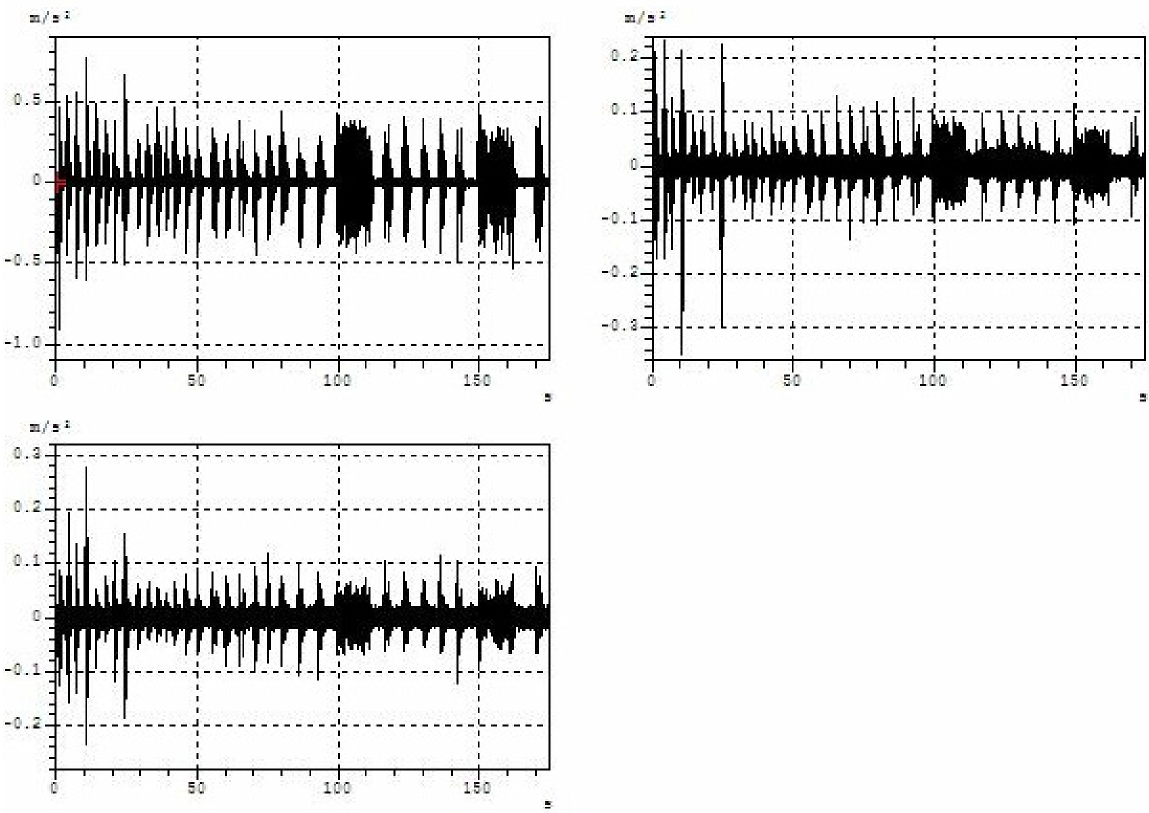


Figure B.10: Time history plots set 2 for the concrete footbridge

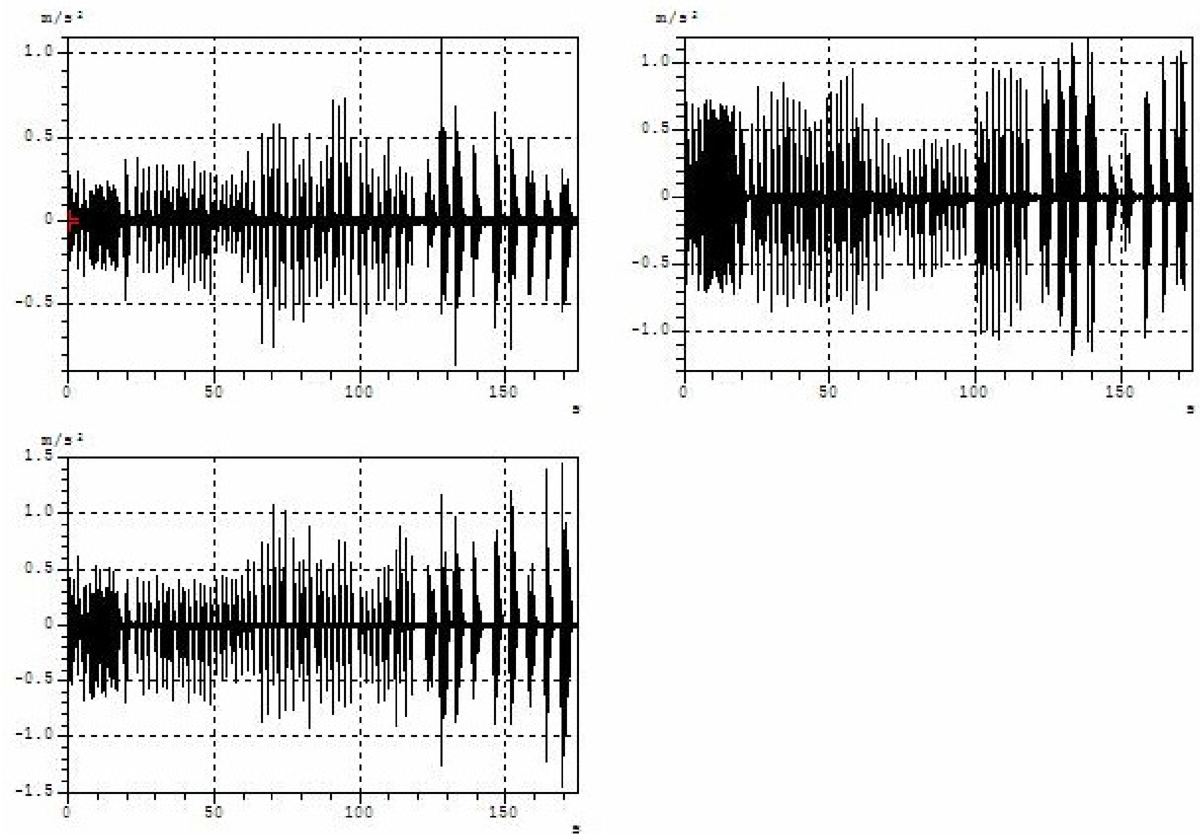


Figure B.11: Time history plots set 3 for the concrete bridge

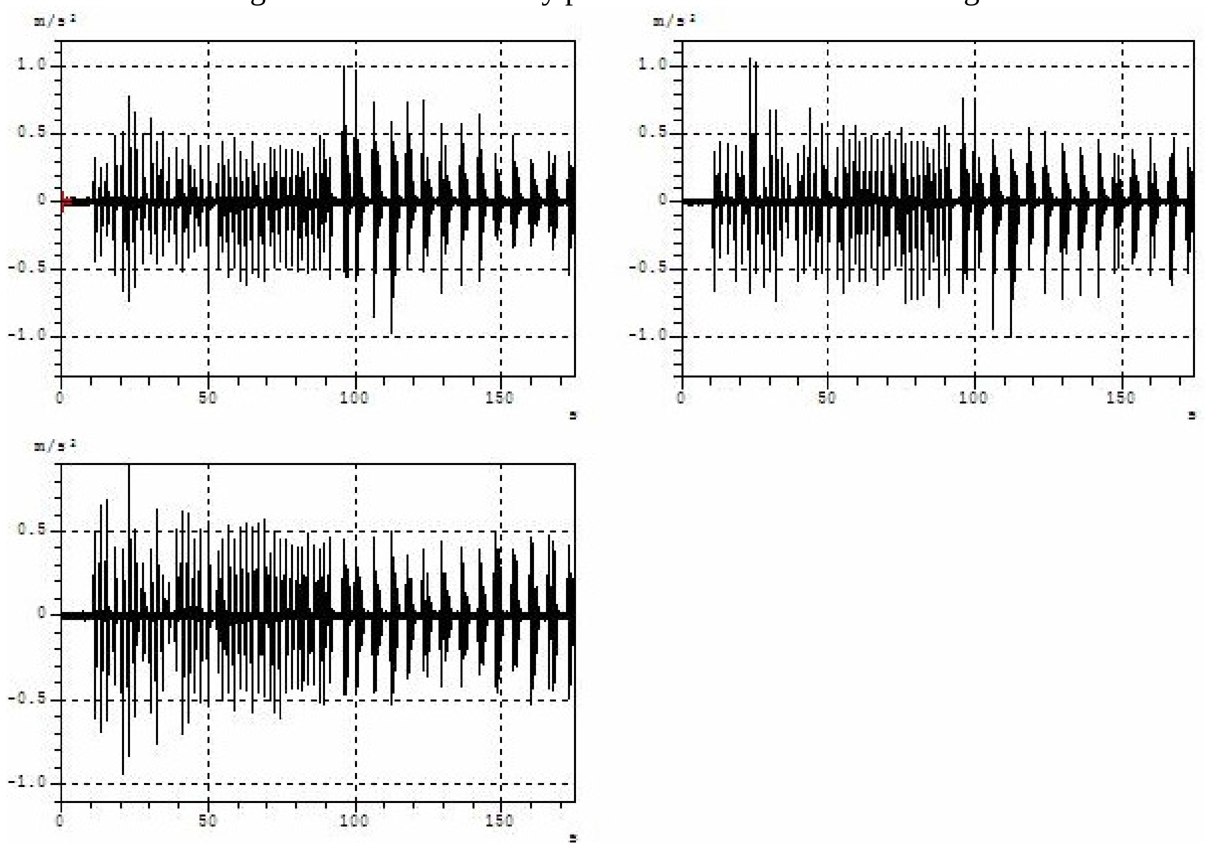


Figure B.12: Time history plots set 4 for the concrete bridge

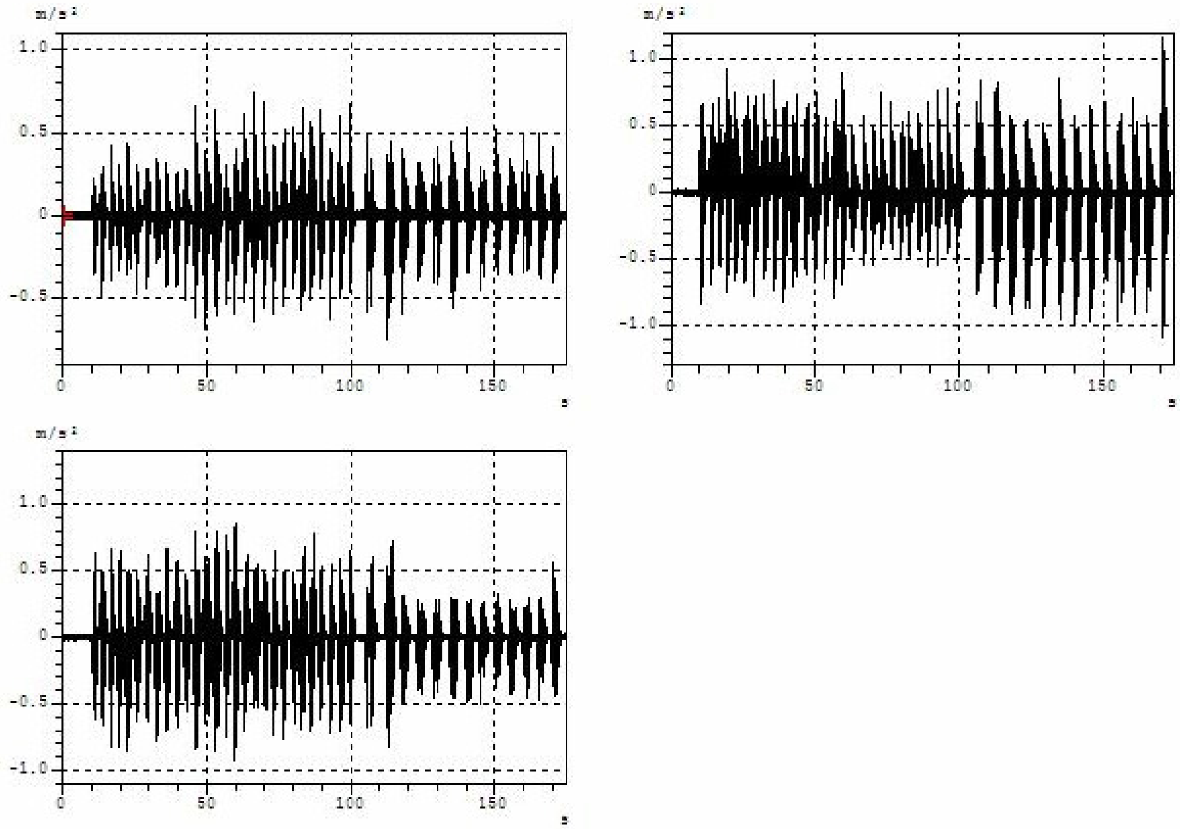


Figure B.13: Time history plots set 5 for the concrete bridge

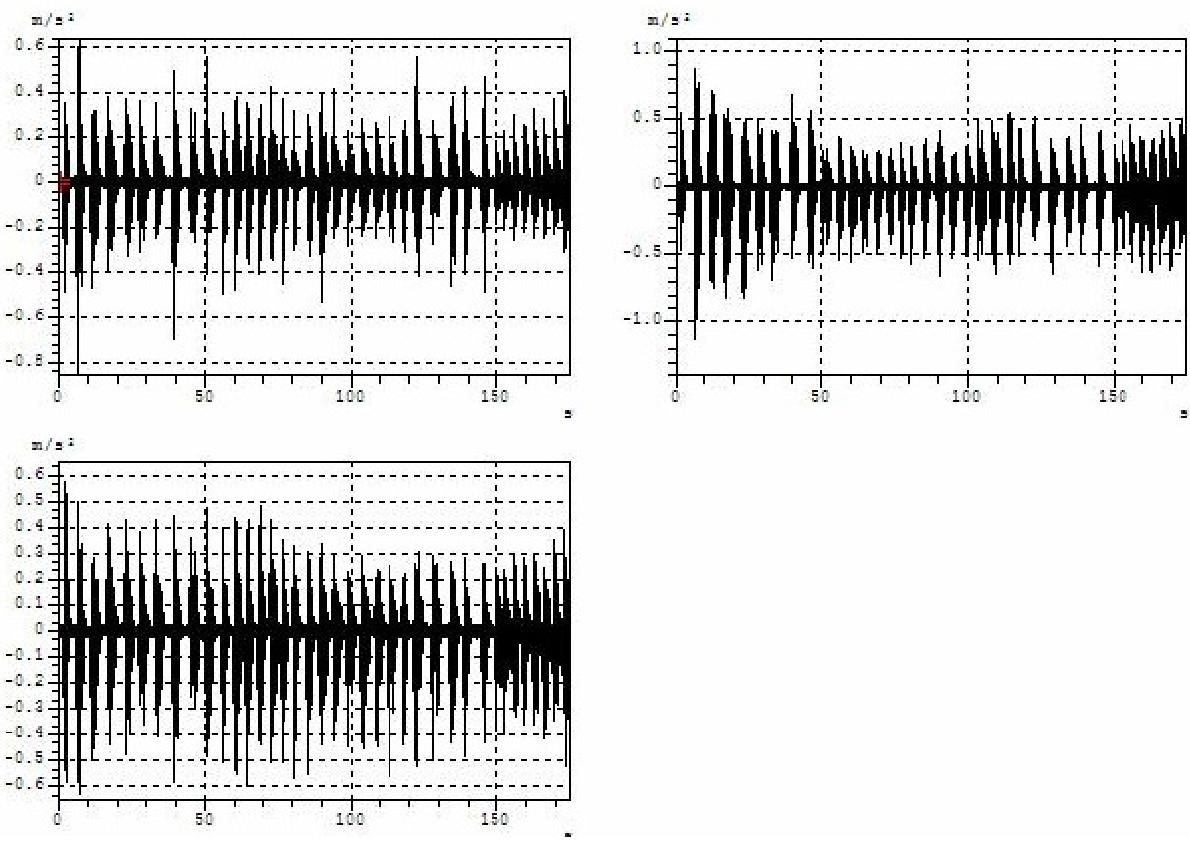


Figure B.14: Time history plots set 6 for the concrete bridge

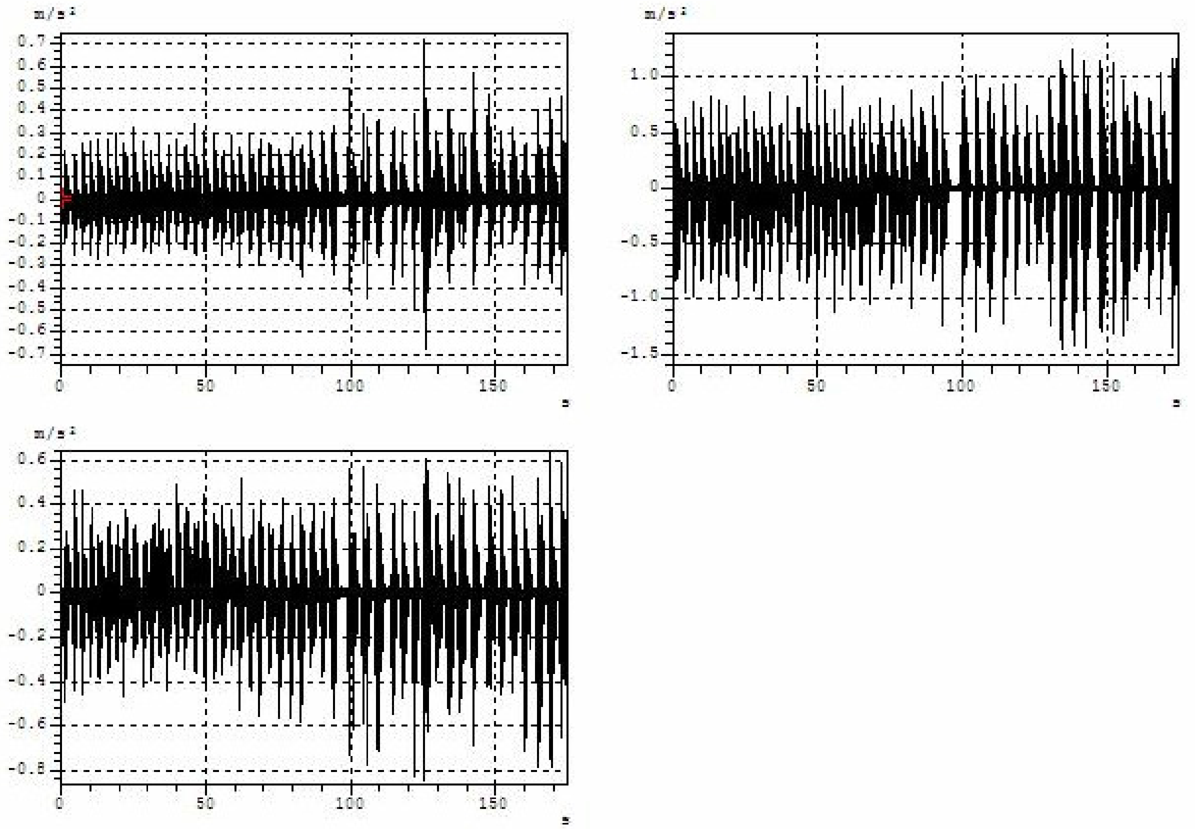


Figure B.15: Time history plots set 7 for the concrete bridge

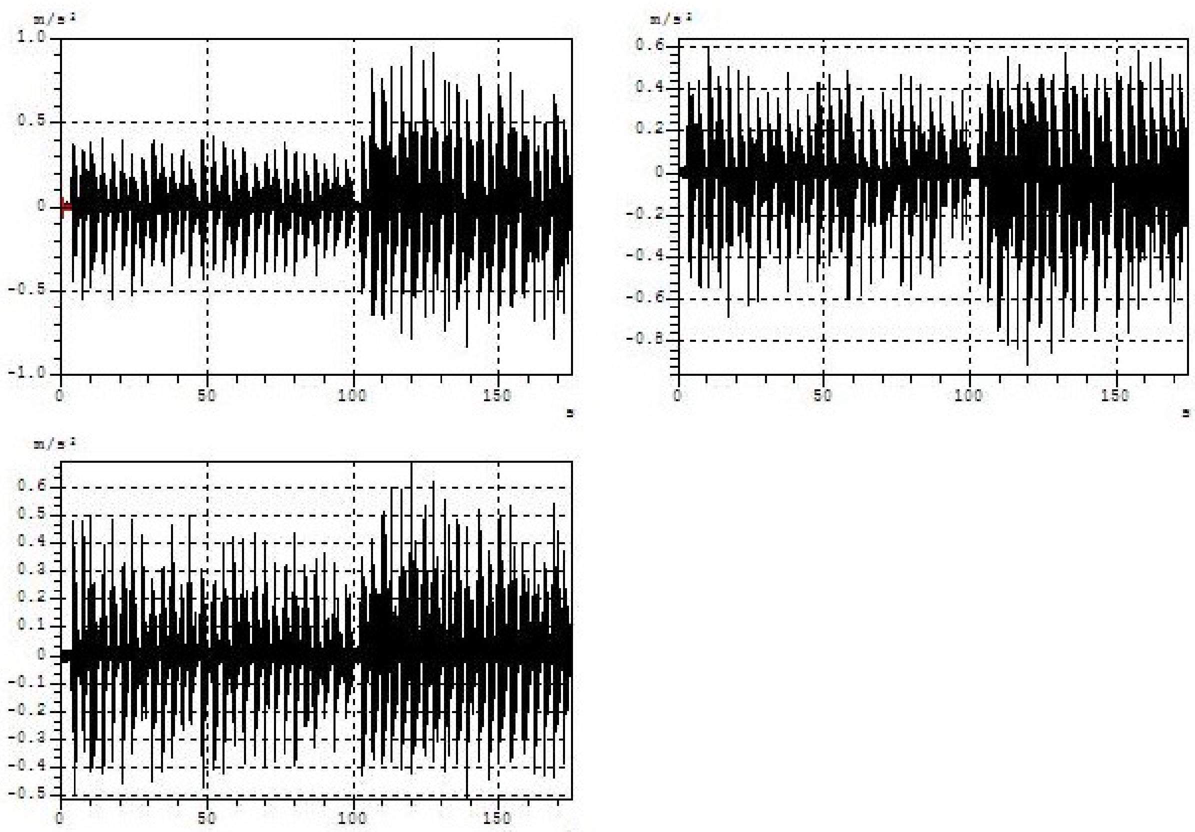


Figure B.16: Time history plots set 8 for the concrete bridge

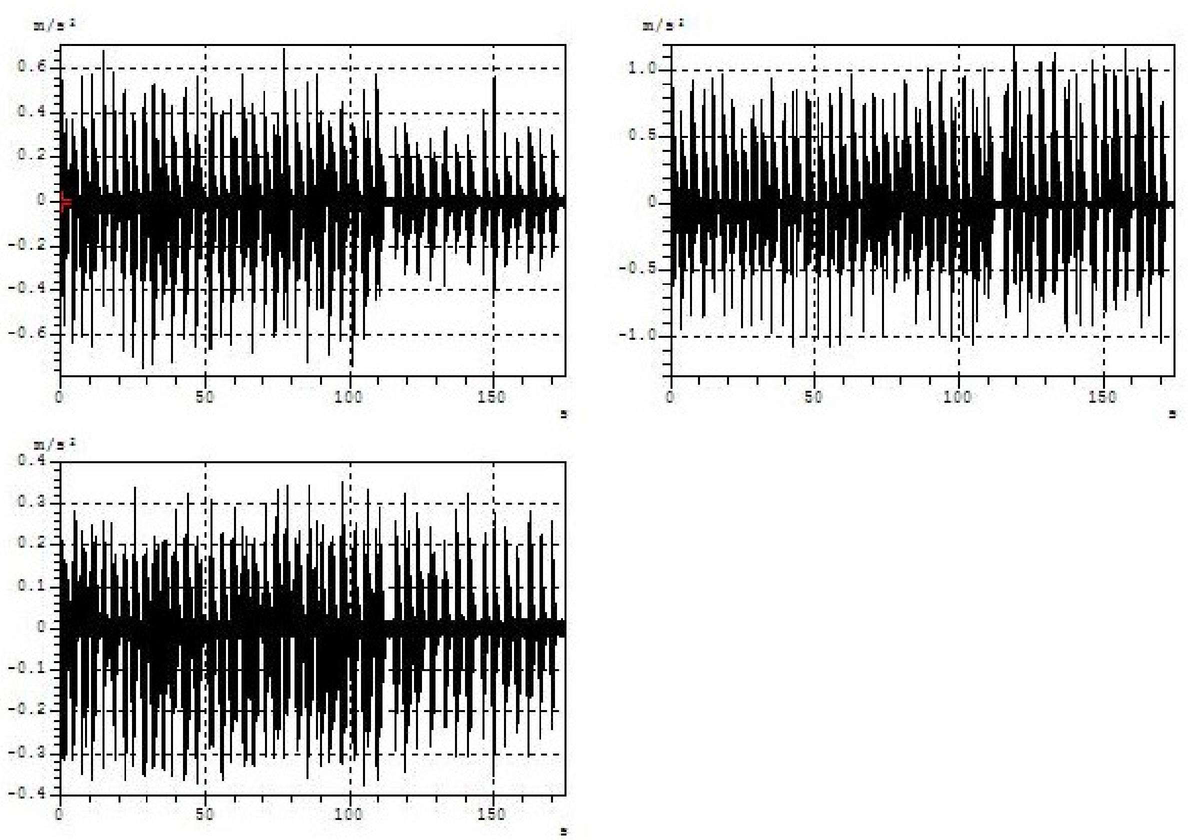


Figure B.17: Time history plots set 9 for the concrete bridge

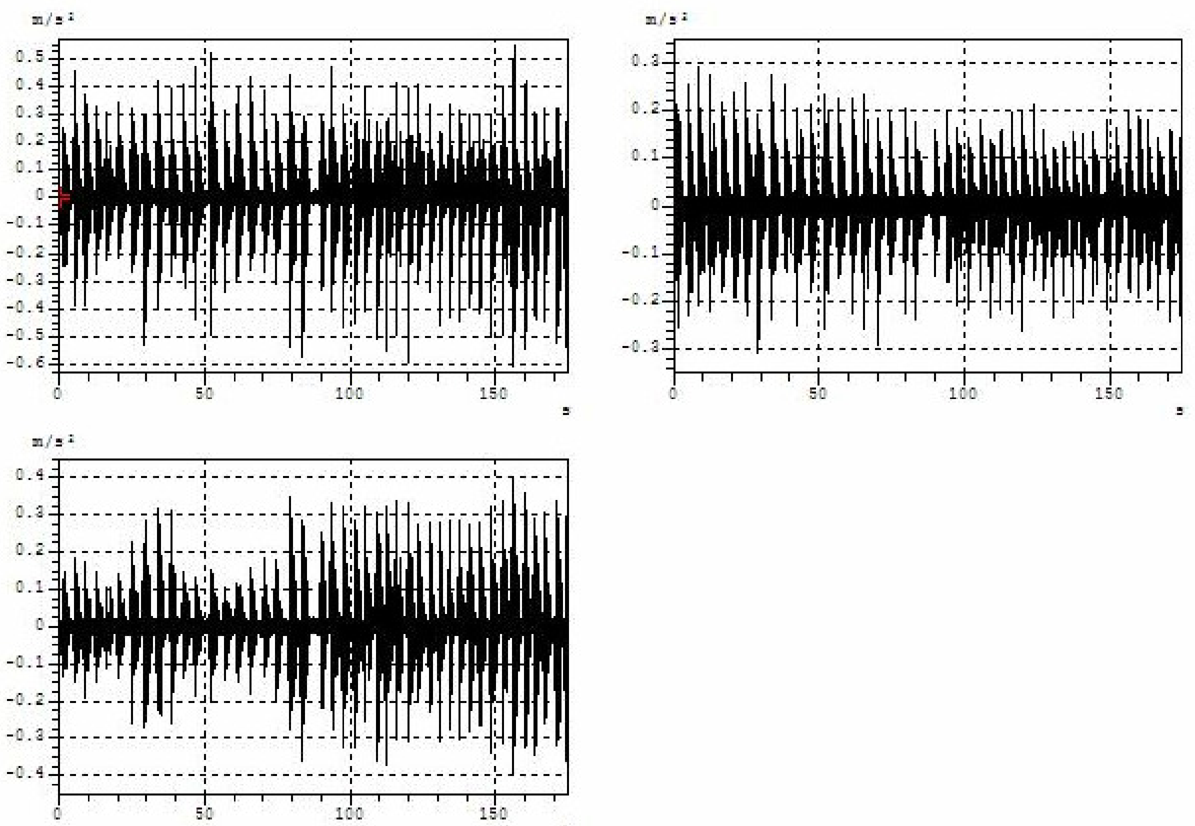


Figure B.18: Time history plots set 10 for the concrete bridge

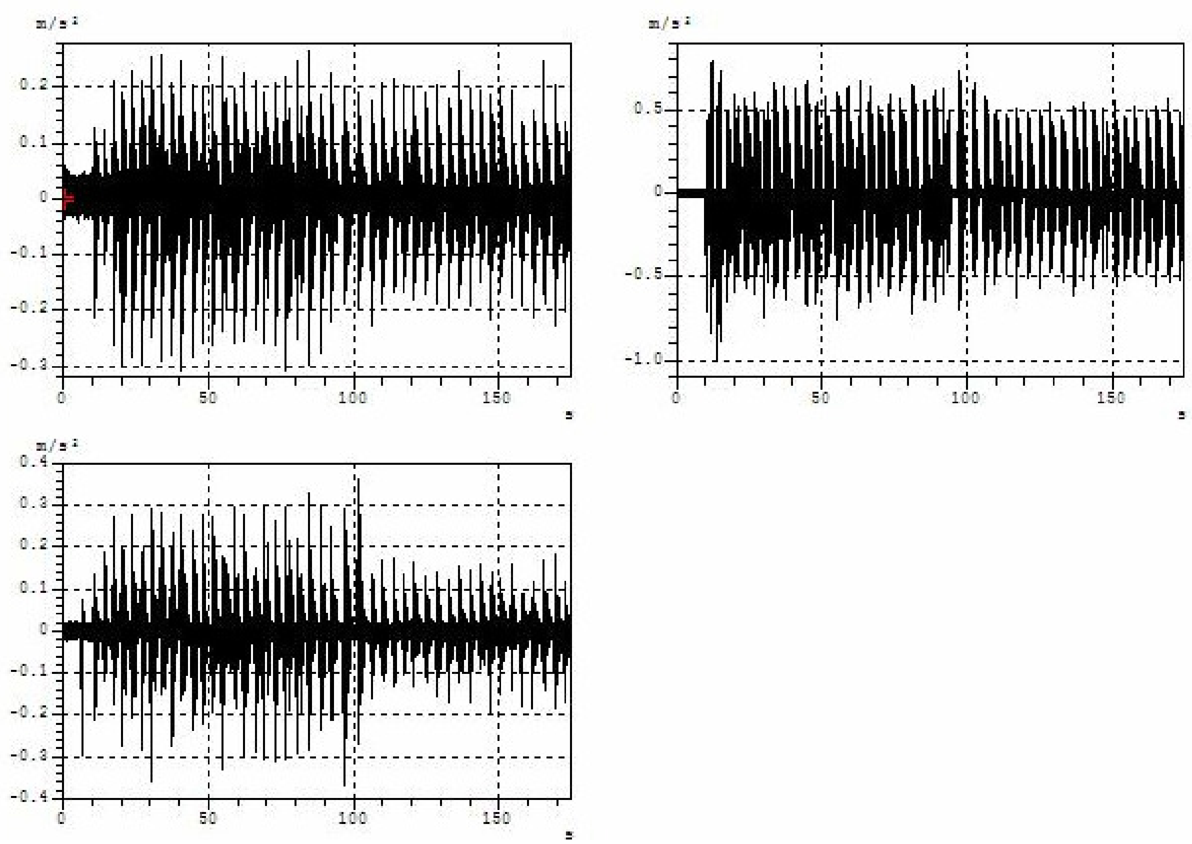


Figure B.19: Time history plots set 11 for the concrete bridge

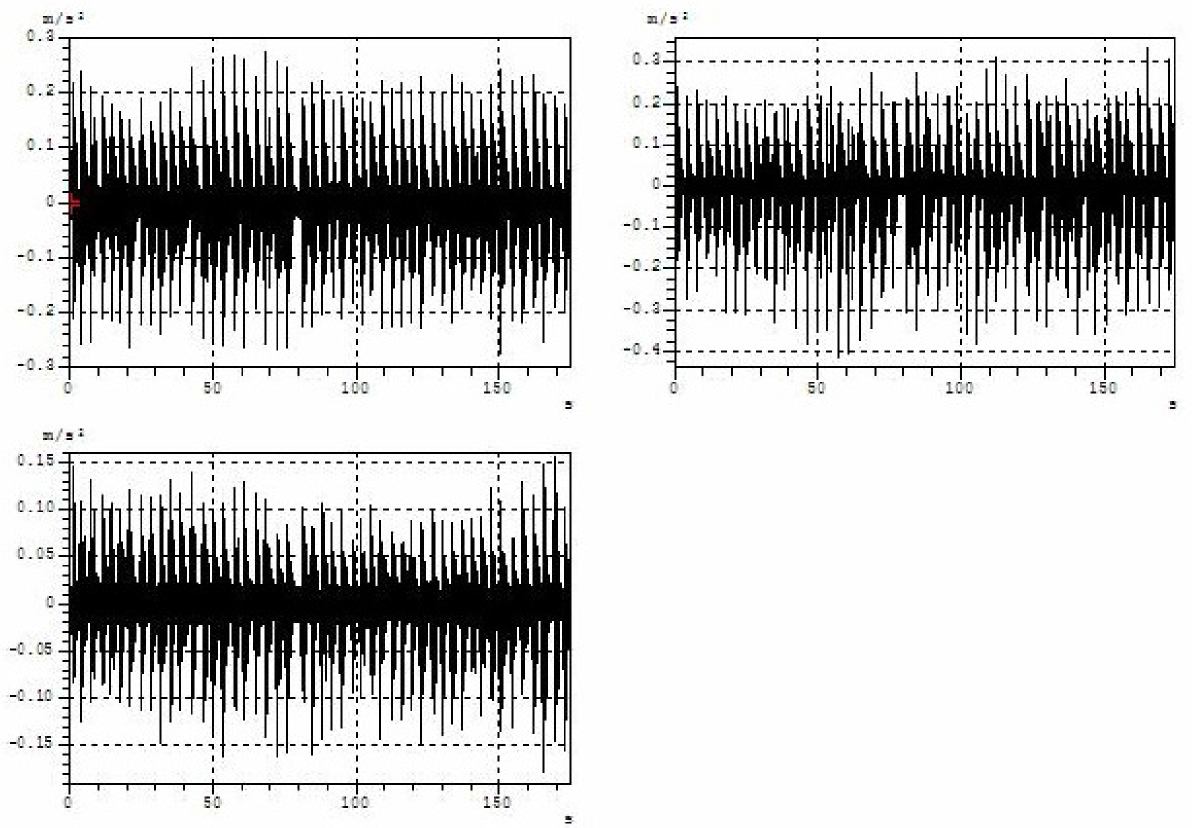


Figure B.20: Time history plots set 12 for the concrete bridge

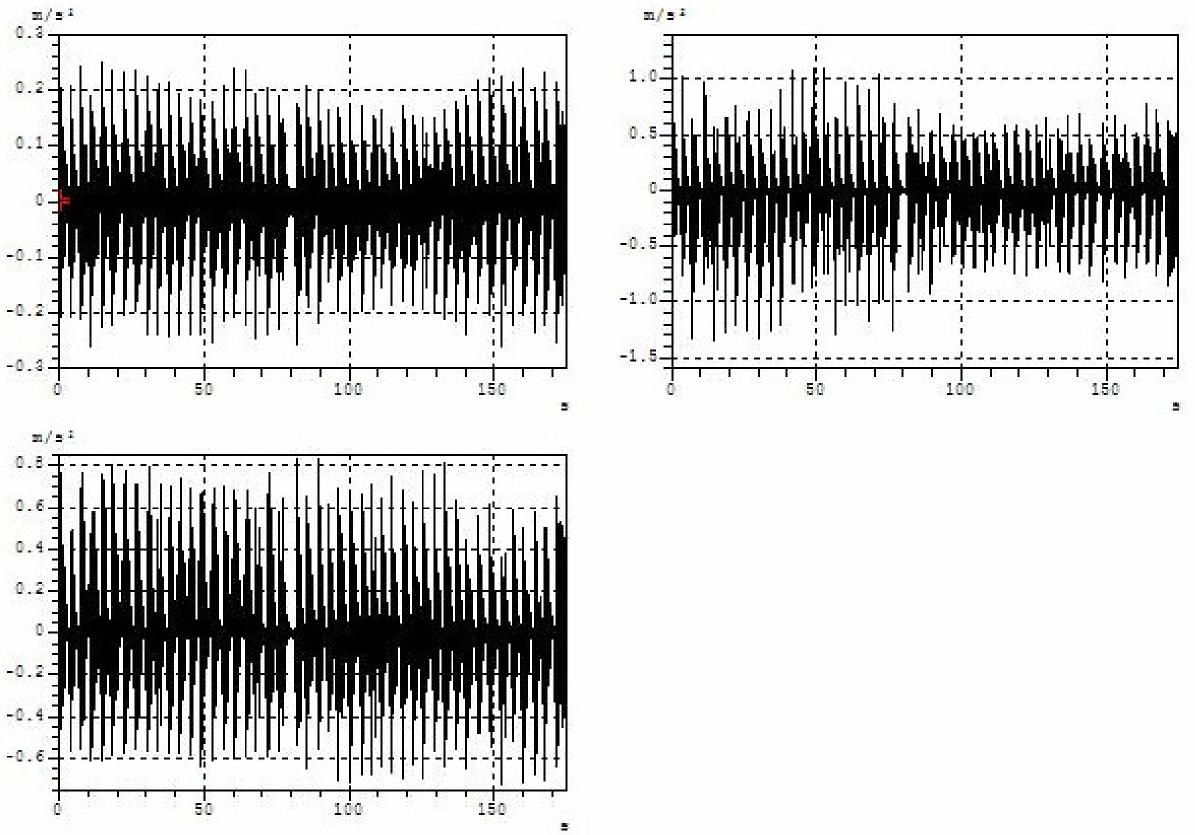


Figure B.21: Time history plots set 13 for the concrete bridge

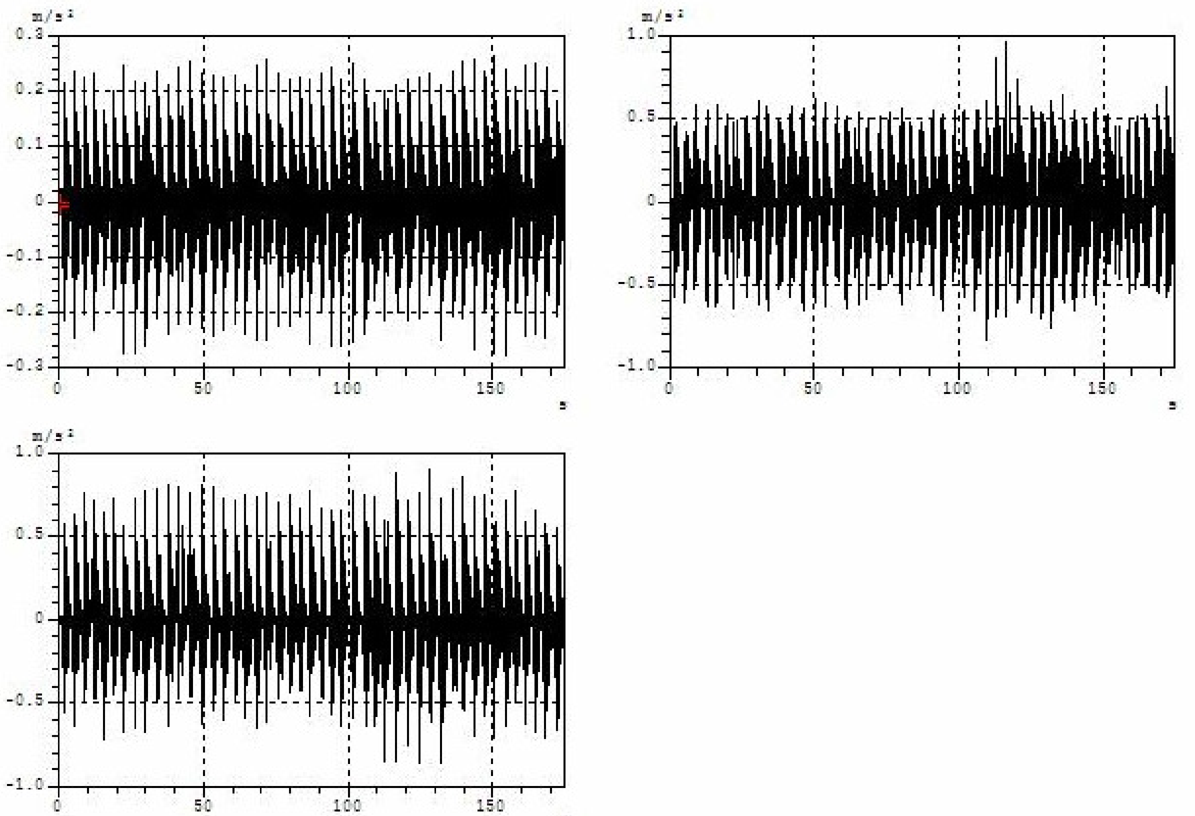


Figure B.22: Time history plots set 14 for the concrete bridge

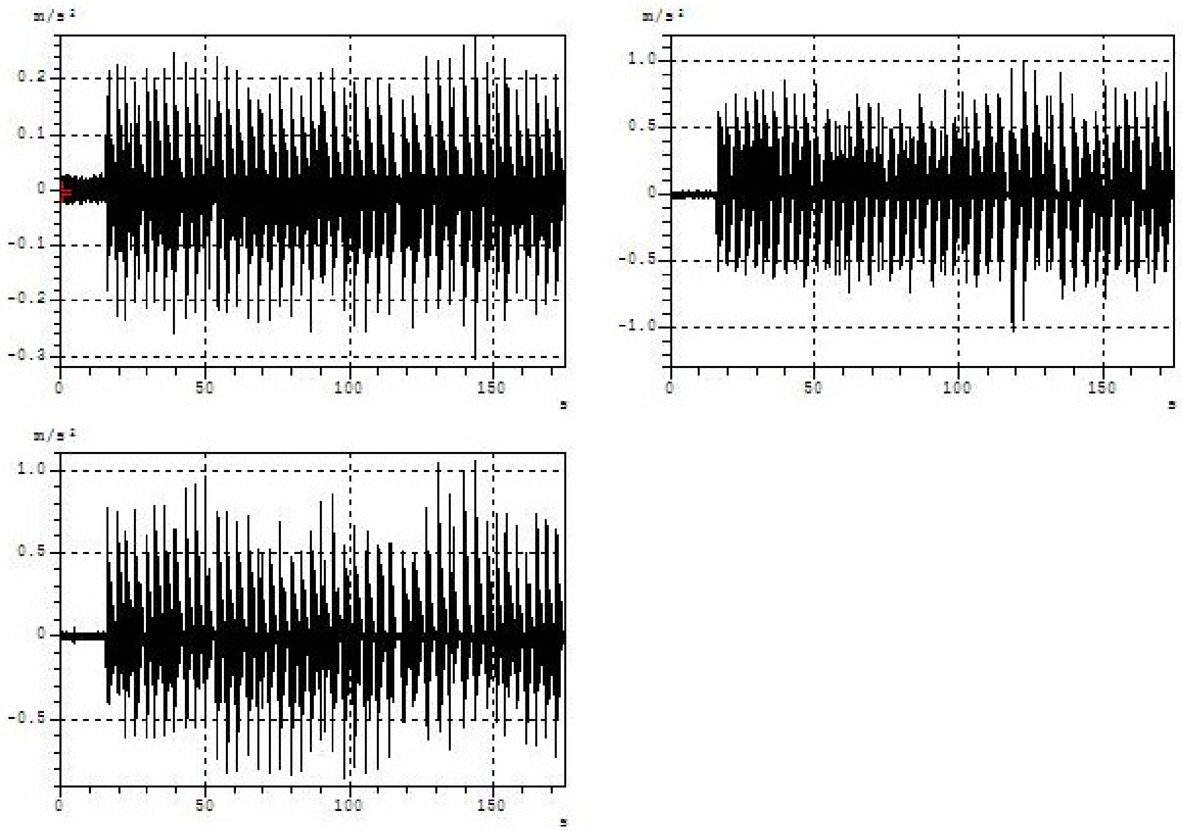


Figure B.23: Time history plots set 15 for the concrete bridge

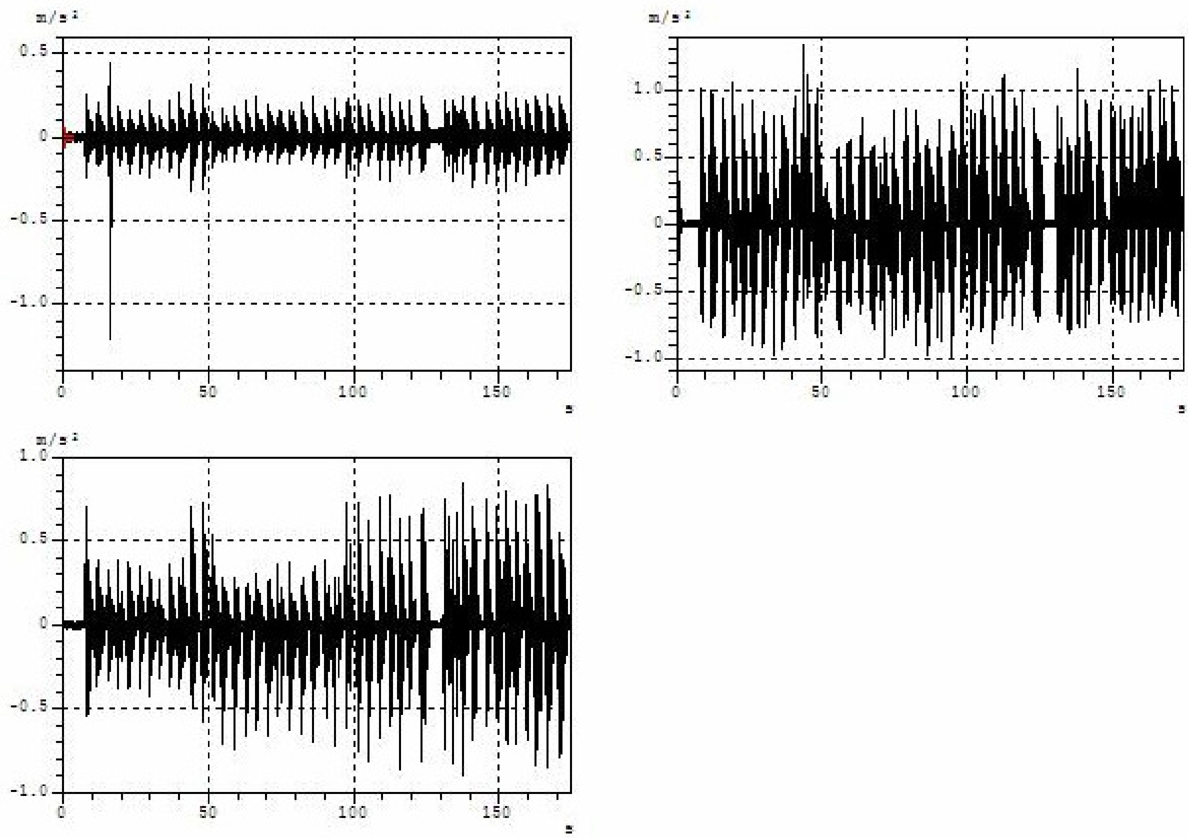


Figure B.24 Time history plots set 16 for the concrete bridge

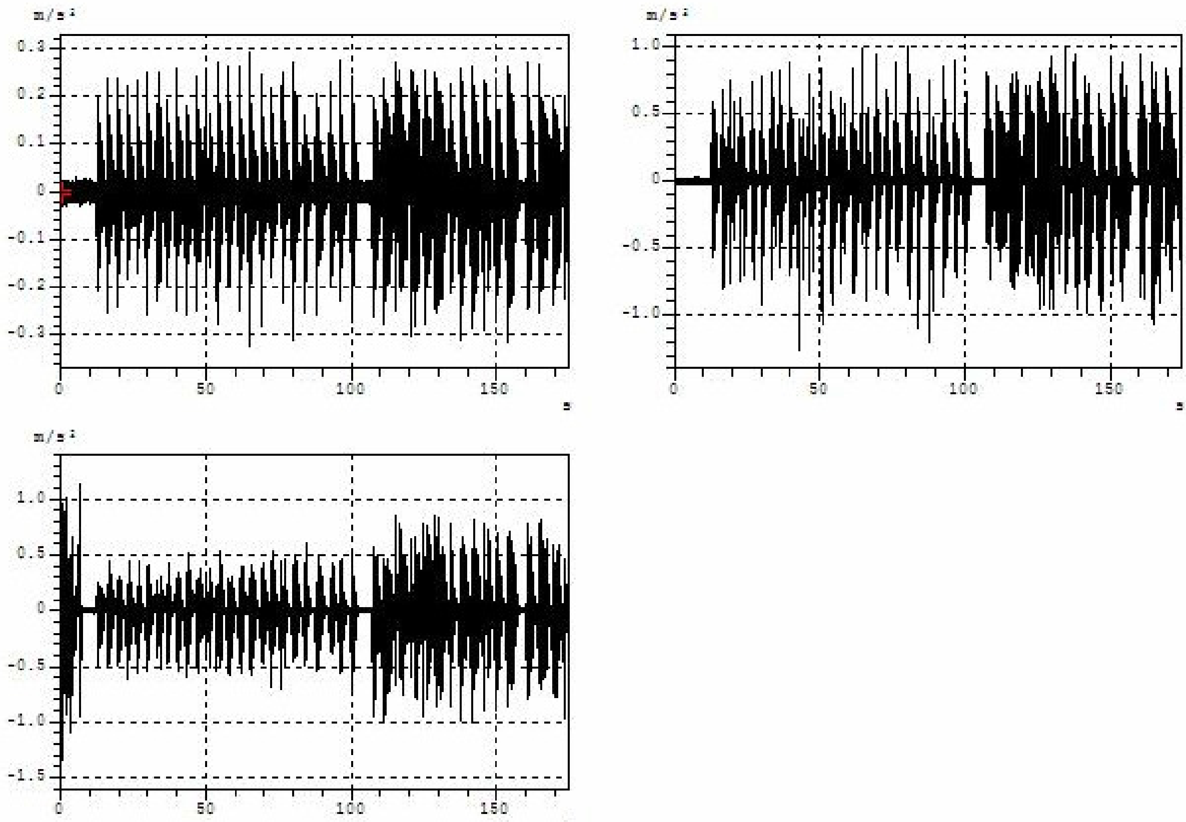


Figure B.25: Time history plots set 17 for the concrete bridge

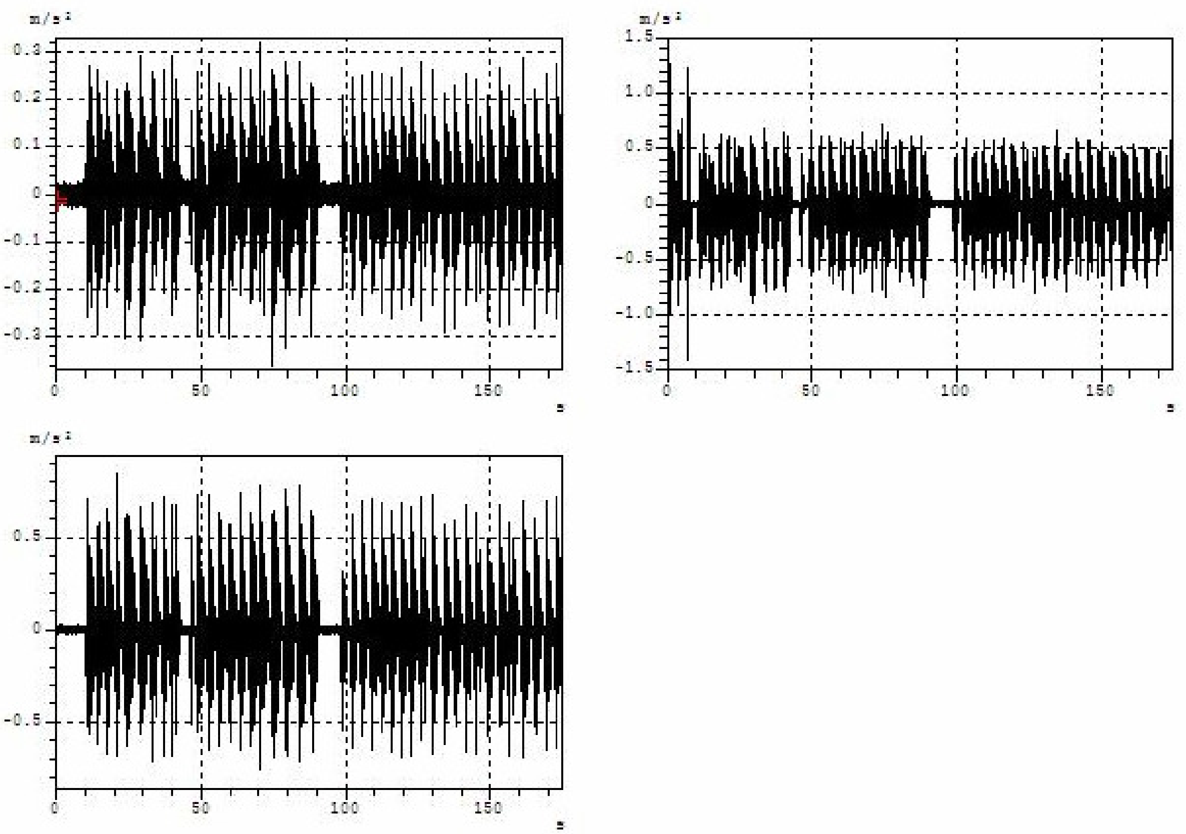


Figure B.26: Time history plots set 18 for the concrete bridge

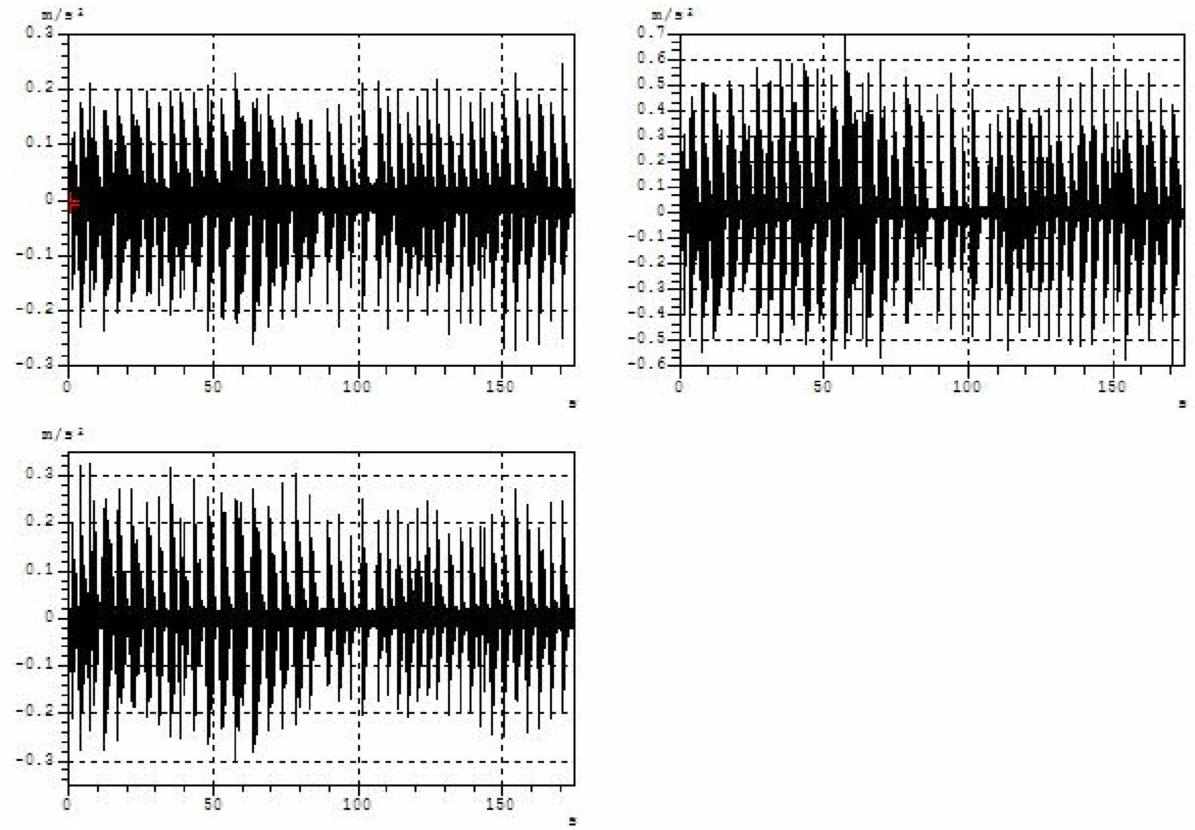


Figure B.27: Time history plots set 19 for the concrete bridge

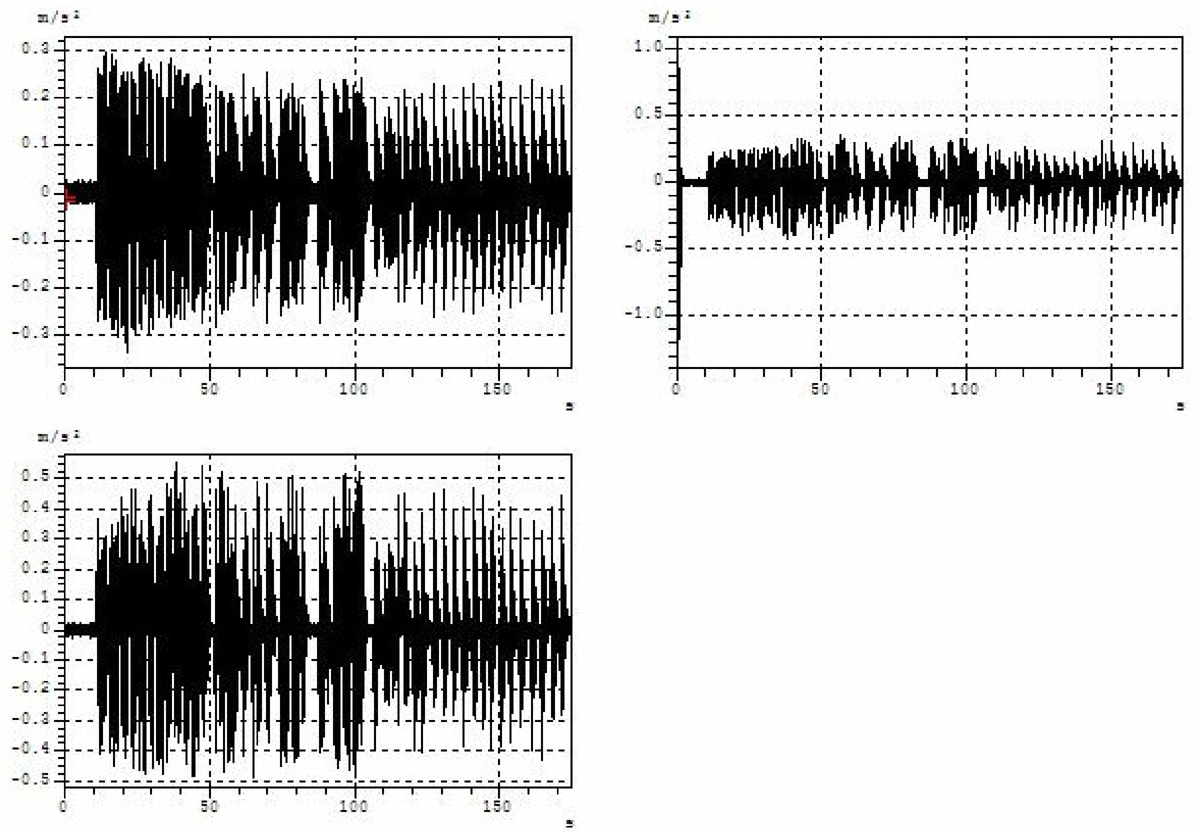


Figure B.28: Time history plots set 20 for the concrete bridge

Appendix C

ANSYS commands for the Staircase Structure

```
/FILNAM,FKM Staircase
/title, FKM Staircase
/prep7
/VUP,1,Z
/VIEW,1,1,-1,1
/replot
!Create geometry of staircase
!Keypoints of staircase
k, 100, 0.0000000000, 0.0000000000, 0.0000000000
k, 101,-0.0328271710, 0.0000000000, 0.2175000000
k, 102, 0.2121728290, 0.0000000000, 0.2175000000
k, 103, 0.1921728290, 0.0000000000, 0.3625000000
k, 104, 0.4571728290, 0.0000000000, 0.3625000000
k, 105, 0.4371728290, 0.0000000000, 0.5125000000
k, 106, 0.7096728290, 0.0000000000, 0.5125000000
k, 107, 0.6896728290, 0.0000000000, 0.6625000000
k, 108, 0.9546728290, 0.0000000000, 0.6625000000
k, 109, 0.9346728290, 0.0000000000, 0.8150000000
k, 110, 1.1996728290, 0.0000000000, 0.8150000000
k, 111, 1.1796728290, 0.0000000000, 0.9675000000
k, 112, 1.4346728290, 0.0000000000, 0.9675000000
k, 113, 1.4146728290, 0.0000000000, 1.1150000000
k, 114, 1.6846728290, 0.0000000000, 1.1150000000
k, 115, 1.6646728290, 0.0000000000, 1.2650000000
k, 116, 1.9246728290, 0.0000000000, 1.2650000000
k, 117, 1.9046728290, 0.0000000000, 1.4200000000
k, 118, 2.1746728290, 0.0000000000, 1.4200000000
k, 119, 2.1521728290, 0.0000000000, 1.5700000000
k, 120, 2.4071728290, 0.0000000000, 1.5700000000
k, 121, 2.3871728290, 0.0000000000, 1.7200000000
k, 122, 2.6521728290, 0.0000000000, 1.7200000000
k, 123, 2.6321728290, 0.0000000000, 1.8725000000
k, 124, 2.8796728290, 0.0000000000, 1.8725000000
k, 125, 2.8602152019, 0.0000000000, 2.0125000000
k, 126, 3.1016728290, 0.0000000000, 2.0125000000
k, 127, 3.1016728290, 0.0000000000, 2.1725000000
k, 128, 4.6716728290, 0.0000000000, 2.1725000000
k, 129, 4.6716728290, 0.0000000000, 2.0125000000
k, 130, 3.9916728290, 0.0000000000, 2.0125000000
k, 131, 3.9916728290, 0.0000000000, 1.8675000000
k, 132, 3.7416728290, 0.0000000000, 1.8675000000
k, 133, 3.7416728290, 0.0000000000, 2.0125000000
k, 134, 3.3916728290, 0.0000000000, 2.0125000000
k, 135, 0.2082644921, 0.0000000000, 0.0000000000
```

```
k, 200, 3.1016728290, 1.4950000000, 2.0125000000
k, 201, 3.1016728290, 1.4950000000, 2.1725000000
k, 202, 2.8816728290, 1.4950000000, 2.1725000000
k, 203, 2.9016728290, 1.4950000000, 2.3300000000
k, 204, 2.6416728290, 1.4950000000, 2.3300000000
k, 205, 2.6616728290, 1.4950000000, 2.4800000000
k, 206, 2.3991728290, 1.4950000000, 2.4800000000
k, 207, 2.4191728290, 1.4950000000, 2.6400000000
k, 208, 2.1716728290, 1.4950000000, 2.6400000000
k, 209, 2.1916728290, 1.4950000000, 2.7825000000
k, 210, 1.9241728290, 1.4950000000, 2.7825000000
k, 211, 1.9441728290, 1.4950000000, 2.9375000000
k, 212, 1.6966728290, 1.4950000000, 2.9375000000
k, 213, 1.7166728290, 1.4950000000, 3.1000000000
k, 214, 1.4641728290, 1.4950000000, 3.1000000000
k, 215, 1.4841728290, 1.4950000000, 3.2525000000
k, 216, 1.2216728290, 1.4950000000, 3.2525000000
k, 217, 1.2416728290, 1.4950000000, 3.4025000000
k, 218, 0.9916728290, 1.4950000000, 3.4025000000
k, 219, 1.0116728290, 1.4950000000, 3.5550000000
k, 220, 0.7466728290, 1.4950000000, 3.5550000000
k, 221, 0.7666728290, 1.4950000000, 3.7025000000
k, 222, 0.5166728290, 1.4950000000, 3.7025000000
k, 223, 0.5366728290, 1.4950000000, 3.8550000000
k, 224, 0.2691728290, 1.4950000000, 3.8550000000
k, 225, 0.2891728290, 1.4950000000, 4.0200000000
k, 226, 0.0316728290, 1.4950000000, 4.0200000000
k, 227, 0.0516728290, 1.4950000000, 4.1775000000
k, 228,-0.2183271710, 1.4950000000, 4.1775000000
k, 229,-0.1983271710, 1.4950000000, 4.3525000000
k, 230,-0.8482031551, 1.4950000000, 4.3525000000
k, 231,-0.8482031551, 1.4950000000, 4.1925000000
k, 232,-0.5913737601, 1.4950000000, 4.1925000000
k, 233, 2.7791998060, 1.4950000000, 2.0125000000
k, 234,-0.2183271710, 1.4950000000, 3.9512230932
k, 300,3.7416728290, 1.26, 0.0
k, 301,3.9916728290, 1.26, 0.0
k, 302,3.9916728290, 1.51, 0.0
k, 303,3.7416728290, 1.51, 0.0
! Area for Stair Landing and beam (Area no 1)
A,126,127,128,129,130,131,132,133,134
! Area for column base (Area no 2)
A,300,301,302,303
! Area for Upper Stairs (Area no 3 & 4)
A,200,201,202,203,204,205,206,207,208,209,210,211,212,213,214,215,216,233
A,216,217,218,219,220,221,222,223,224,225,226,227,228,234,233
```

```
! Area for Lower Stairs (Area no 5 & 6)
A,100,101,102,103,104,105,106,107,108,109,110,111,112,113,114,115,116,135
A,116,117,118,119,120,121,122,123,124,125,126,134,135

! Combine Area 5 & 6 = 7
AADD,5,6

! Combine Area 3 & 4 = 5
AADD,3,4
APLOT

! Create Stairs of length 1.275m in the y-dir by extrude area (Area No 1,2,3 & 4) into
Volume

! VEXT, NA1, NA2, NINC, DX, DY, DZ, RX, RY, RZ
VEXT, 5, 7, 2, 0, 1.275, 0, 0, 0, 0

! Create Landing of length 2.77m in the Y-dir
VEXT,1, , ,0,2.77,0,,,,

! Create Column of length 1.8675m in the z-dir
VEXT,2, , ,0,0,1.8675,,,,

! Combine all the volumes of straicase
!VGLUE,ALL

! Material Properties of staircase
ET,1,SOLID92

!*
!*
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,EX,1,,24.5e9
MPDATA,PRXY,1,,.2
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,DENS,1,,2400
! Boundary condition for staircase
DA,2,ALL
DA,18,ALL
DA,50,ALL
!EMATWRITE,YES
VSEL,S,,,1,4,1
VADD,ALL
```

```
! Meshing of staircase
SMRTSIZE,5
MSHKEY,0
VMESH,ALL

/SOLU
!Modal solution (20 modes)
ANTYPE,2
MODOPT,SUBSP,20
OUTRES,ALL,ALL,
MXPAND,20
SOLVE
```

ANSYS commands for the Timber bridge

```
/FILNAM,UTMTimberBridge
/title, UTM Timber Bridge
/prep7
/VUP,1,Z
/VIEW,1,1,-1,1
/replot

!Define Element Types and Properties
ET,1,BEAM4
ET,2,shell63

!Define real constants

!Steel Girder
R,1,4.775e-5,8.747e-5,7.4899e-6,.315,.165, ,
RMORE, ,1.0825e-7, , , , ,

!RHS 150x85x10
R,2,4.3e-3,4.7015e-6,1.2006e-5,85,150, ,
RMORE, ,1.02723e-5, , , , ,

!RHS 150x75x10
R,3,4.1e-3,1.1024e-5,3.471e-6, , , ,
RMORE, ,8.4769e-6, , , , ,

!Timber Plank assumed as Slab
R,4,0.04
```

```
!Steel Section
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,EX,1,,200e9
MPDATA,PRXY,1,,.3
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,DENS,1,,7850

!Timber
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,EX,2,,19e9
MPDATA,PRXY,2,,0.2
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,DENS,2,,980

!Create geometry
!Keypoints

K,1,0,0,0,
K,2,0,0.4975,0, !Support
K,3,0,3.5775,0,
K,4,0,7.1375,0,
K,5,0,10.6625,0,
K,6,0,13.7425,0, !Support
K,7,0,14.2400,0,

K,8,0.8425,0,0,
K,9,0.8425,0.4975,0,
K,10,0.8425,3.5775,0,
K,11,0.8425,7.1375,0,
K,12,0.8425,10.6625,0,
K,13,0.8425,13.7425,0,
K,14,0.8425,14.2400,0,

K,15,1.685,0,0,
K,16,1.685,0.4975,0, !Support
K,17,1.685,3.5775,0,
K,18,1.685,7.1375,0,
K,19,1.685,10.6625,0,
K,20,1.685,13.7425,0, !Support
K,21,1.685,14.2400,0,

!k,22,0,0.1025,0
!k,23,1.85,0.1025,0
```

```
!Line Definition

numstr,line,101
l,1,2
l,2,3
l,3,4
l,4,5
l,5,6
l,6,7
l,15,16
l,16,17
l,17,18
l,18,19
l,19,20
l,20,21

numstr,line,201
l,8,9
l,9,10
l,10,11
l,11,12
l,12,13
l,13,14

!numstr,line,301
!l,1,8
!l,8,15
!l,7,14
!l,14,21

numstr,line,401
l,1,8
l,8,15
l,2,9
l,9,16
l,3,10
l,10,17
l,4,11
l,11,18
l,5,12
l,12,19
l,6,13
l,13,20
l,7,14
l,14,21
```

```
!Assign Real Constants and Material Properties

lsel,s,,,101,112,1
latt,1,1,1

lsel,s,,,201,206,1
latt,1,2,1

lsel,s,,,403,412,1
latt,1,3,1

!lsel,s,,,301
!latt,2,4,1

Allsel

!Generate shells

numstr,area,2001
al,101,403,201,401 ! count+100,count+402,count+200,count+400
al,102,405,202,403
al,103,407,203,405
al,104,409,204,407
al,105,411,205,409
al,106,413,206,411

al,201,404,107,402
al,202,406,108,404
al,203,408,109,406
al,204,410,110,408
al,205,412,111,410
al,206,414,112,412
asel,s,,,2001,2012
aatt,2,4,2

allsel
lsel,s,,,401,414
lesize,all,,,3
lsel,s,,,102,105
lsel,a,,,108,111
lesize,all,,,10
lsel,s,,,202,205
lesize,all,,,10
```

```
lsel,s,,,101
lsel,a,,,201
lsel,a,,,107
lsel,s,,,106
lsel,a,,,206
lsel,a,,,112
lesize,all,,,2

lmesh,all
amesh,all

!Boundary Condition
ksel,s,,,2
ksel,a,,,6
ksel,a,,,16
ksel,a,,,20
dk,all,all,0

/SOLU
!Modal solution (20 modes)
ANTYPE,2
MODOPT,SUBSP,20
OUTRES,ALL,ALL,
MXPAND,20

allsel
solve
```

ANSYS commands for the Concrete Bridge

```
/FILNAM,Bridge
/title, GerbangBridge
/prep7
/VUP,1,Z
/VIEW,1,1,-1,1
/replot

!Bridge Geometry
Ld = 10.8
Bd = 0.5
Xd = 0.75
Hd = 0.5
Lb = 10.8
Bb = 0.5
Hb = 0.528
Sd = 0.013
BTB= 0.513
```

```
! Create geometry

! Keypoints

K,1,0,0,0,
K,2,Bb,0,0,
K,3,0,0,Hb,
K,4,Bb,0,Hb,
K,5,-Xd,0,Hb,
K,6,-Xd,0,Hb+Hd,
K,7,0,0,Hb+Hd,
K,8,Bd,0,Hb+Hd,
K,9,Bd+Sd,0,Hb,
K,10,Bd+Sd,0,Hb+Hd,

! Create Areas
A,1,2,4,3 !Area no 1
A,5,3,7,6 !Area no 2
A,3,4,8,7 !Area no 3
A,4,9,10,8 !Area no 4

! Create Volume
VEXT,1,,1,0,Lb,0
VEXT,2,4,1,0,Ld,0

! Generate Volume
VGEN,39,1, , ,BTB, , , ,1
VGEN,39,3, , ,BTB, , , ,1
VGEN,38,4, , ,BTB, , , ,1
VGEN,2,2, , ,20.744, , , ,1

! Structural element
ET,1,SOLID185
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,EX,1,,24.5e9
MPDATA,PRXY,1,,.2
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,DENS,1,,2400
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,EX,2,,30.1e9
MPDATA,PRXY,2,,.2
MPTEMP,,,,,,,,
MPTEMP,1,0
MPDATA,DENS,2,,2400
```

```
VGLUE,ALL

! select element atrribute
VSEL,s,,,119,122
VATT,1,,1

VSEL,s,,,124,198
VATT,1,,1

VSEL,s,,,1
VATT,2,,1

VSEL,s,,,123
VATT,2,,1

VSEL,s,,,6,42
VATT,2,,1

Allsel
! Boundary condition
ASEL,s,,,1,5,1
DA,ALL,ALL

ASEL,s,,,29,245,6
DA,ALL,ALL

ASEL,s,,,722,866,4
DA,ALL,ALL

ASEL,s,,,876,1020,4
DA,ALL,ALL

ASEL,s,,,714,720,6
DA,ALL,ALL

ASEL,s,,,872,873,1
DA,ALL,ALL

ASEL,s,,,30,246,6
DA,ALL,ALL

ASEL,s,,,723,867,4
DA,ALL,ALL

ASEL,s,,,709,715,6
DA,ALL,ALL

ASEL,s,,,711,713,2
DA,ALL,ALL
```

```
ASEL,s,,,713,,1
DA,ALL,ALL

ASEL,s,,,721,,1
DA,ALL,ALL

ASEL,s,,,877,1021,4
DA,ALL,ALL

allsel
! Define element divisions

! side
LESIZE,5,,,3
LESIZE,1568,,,3
LESIZE,3,,,2
LESIZE,1412,,,2

! width
LESIZE,4,,,2
LESIZE,6,,,2
LESIZE,12,,,2
LESIZE,1418,,,2
LESIZE,1397,,,2
LESIZE,485,,,2

! small width
LESIZE,13,,,1
LESIZE,29,,,8
LESIZE,26,,,8
LSEL,s,,,55,487,12
LESIZE,ALL,,,2
LSEL,s,,,1421,1565,4
LESIZE,ALL,,,2
LSEL,s,,,54,486,12
LESIZE,ALL,,,2
LSEL,s,,,1574,1718,4
LESIZE,ALL,,,1

Allsel

! Meshing for bridge
MSHAPE,0,3d !Mapped mesh
MSHKEY,1
VMESH,all
MSHKEY,0
```

```
/SOLU
! Modal solution (20 modes)
ANTYPE,2
MODOPT,SUBSP,20
OUTRES,ALL,ALL,
MXPAND,20
SOLVE
```

Plots of Tested Structures' Boundary Condition

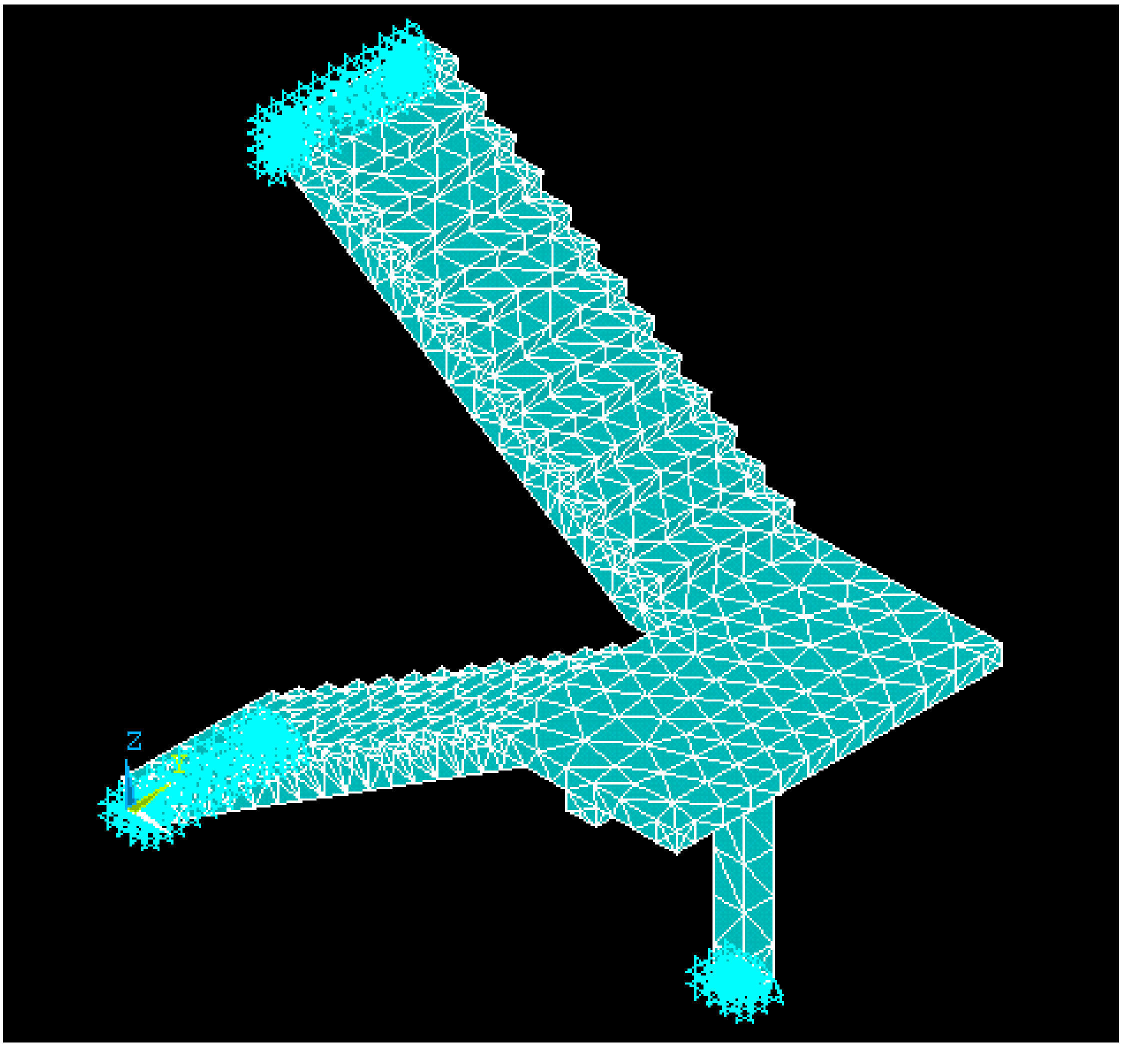


Figure C.1: Boundary condition of the staircase

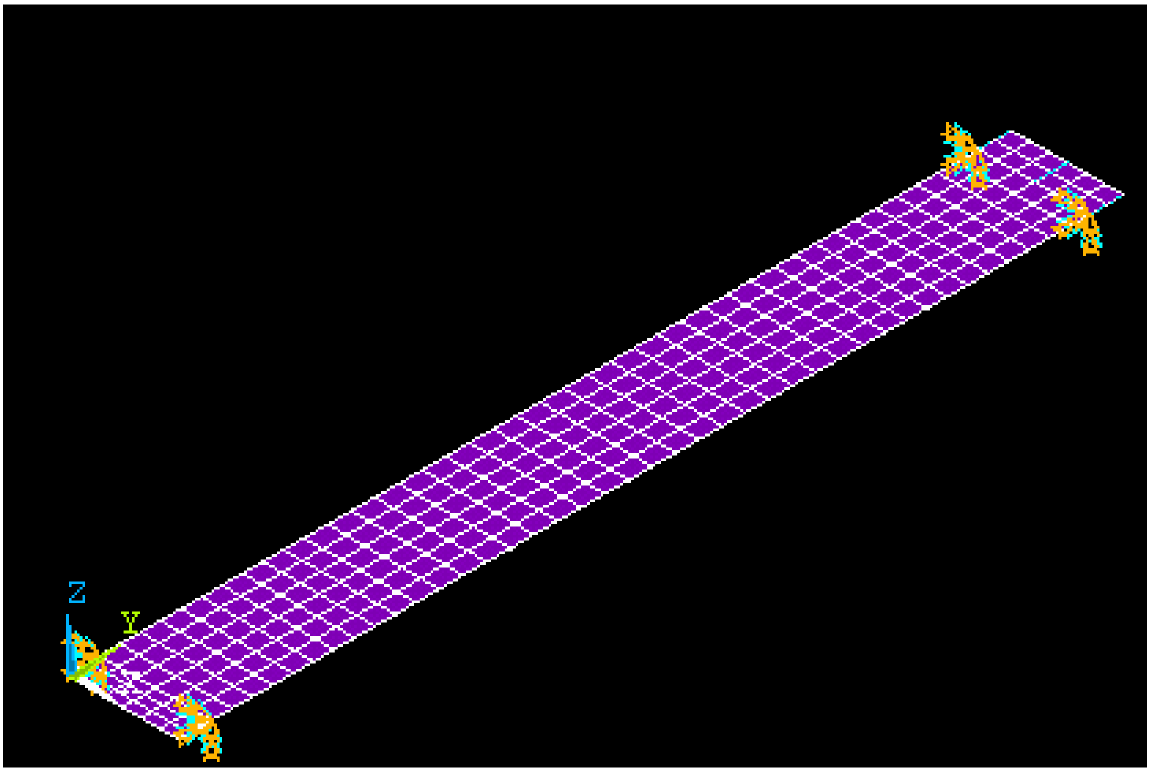


Figure C.2: Boundary condition of the timber footbridge

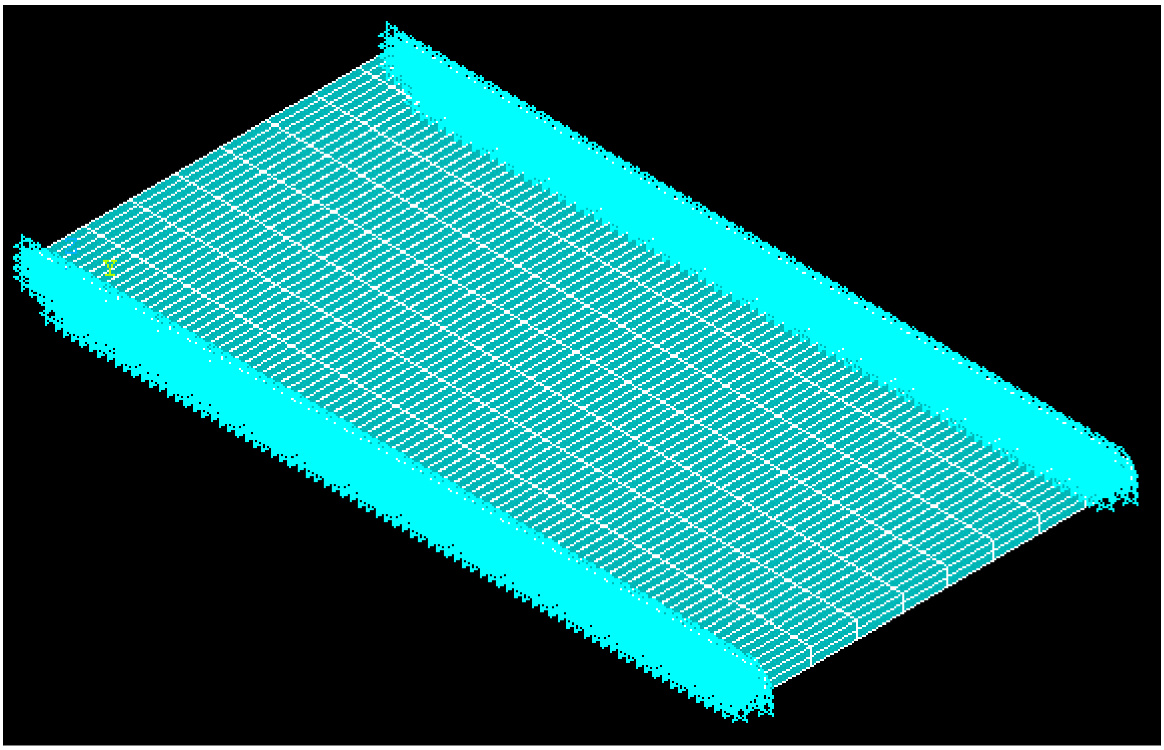
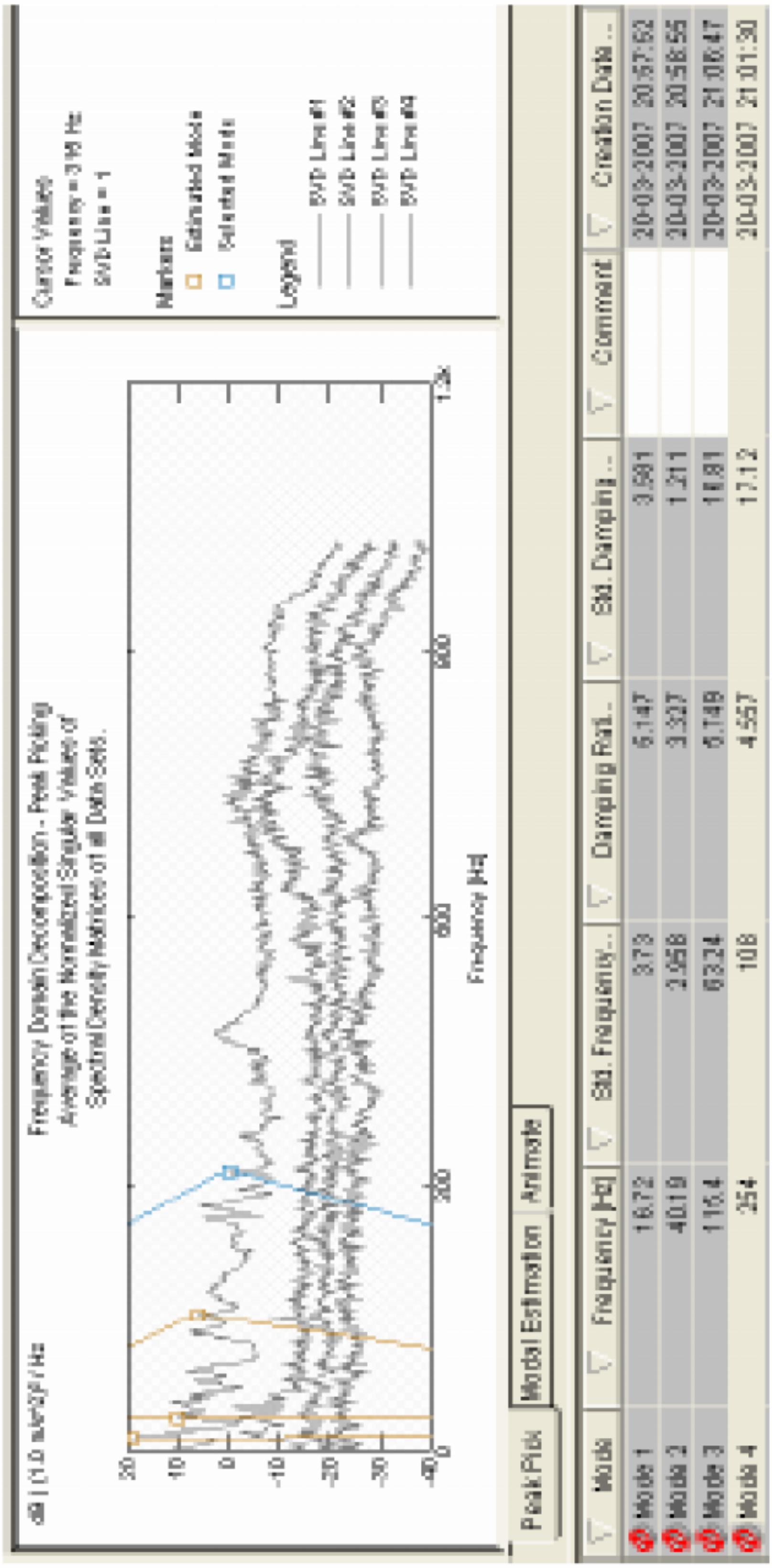


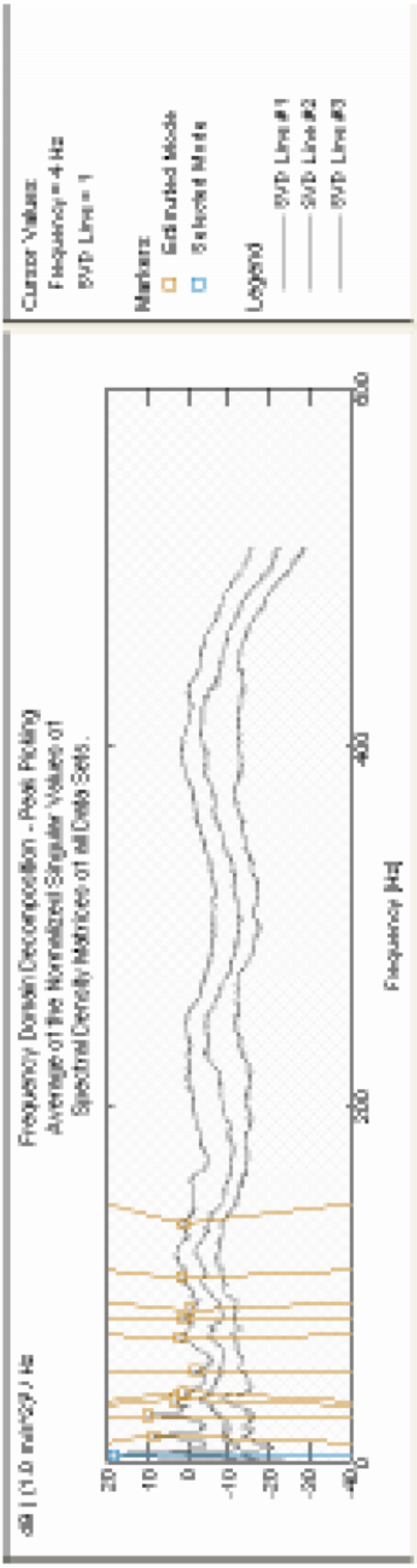
Figure C.3: Boundary condition of the concrete bridge

Appendix D

Peak Picking SV Plot for the Staircase Structure



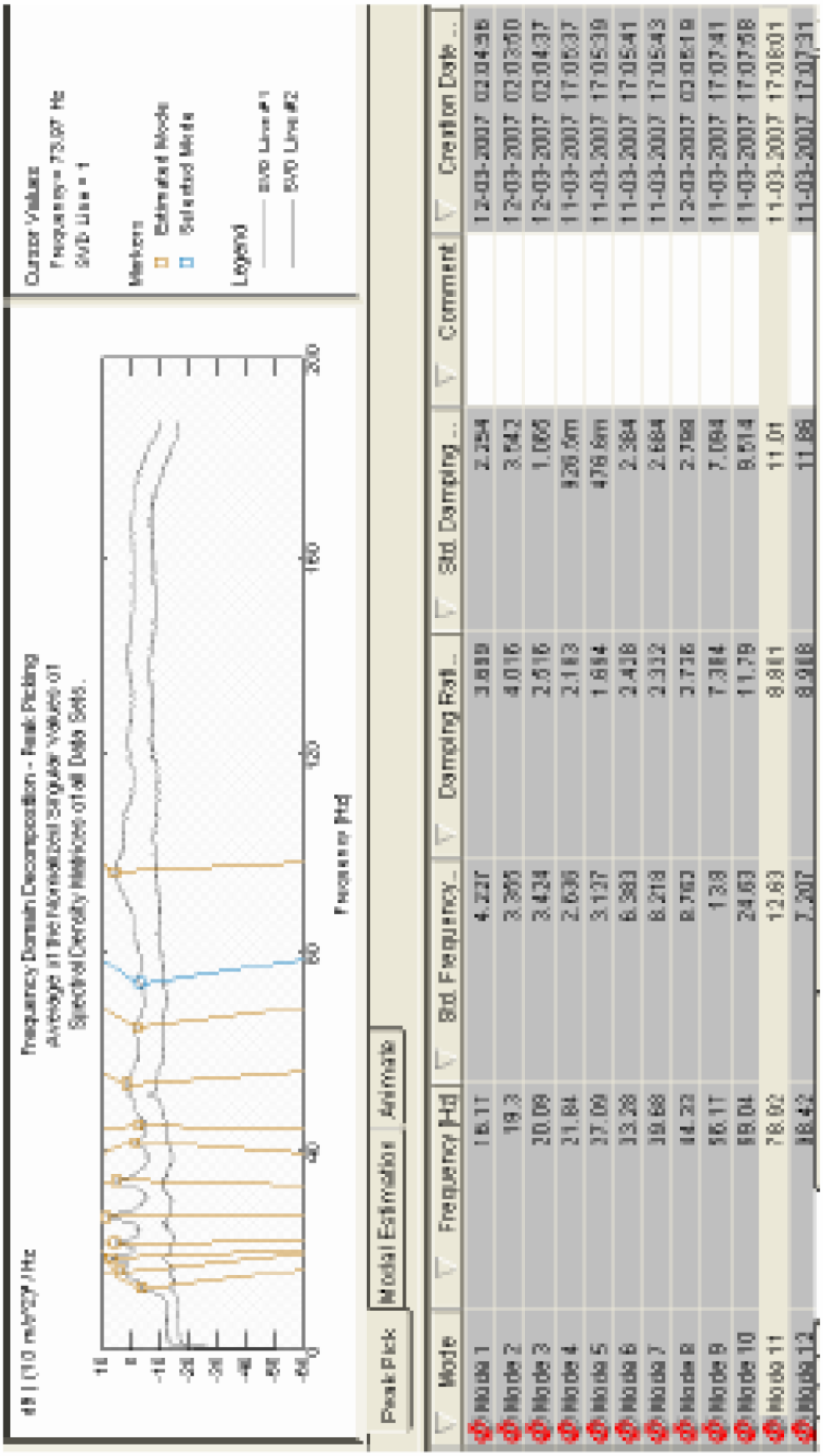
Peak Picking SV Plot for the Timber Footbridge



Peak Pick | Modal Estimation | Animate

Mode	Frequency [Hz]	Std. Frequency...	Damping Rat...	Std. Damping...	Comment	Creation Date ...
Mode 1	4.157	272.4m	11.16	2.105		07-03-2007 17:51:48
Mode 2	11.01	4.874	3.579	5.905		07-03-2007 17:52:04
Mode 3	25.21	9.497	5.408	4.215		07-03-2007 17:52:14
Mode 4	31.32	11.22	4.113	2.504		07-03-2007 17:53:46
Mode 5	33.49	12.45	6.007	2.859		07-03-2007 17:54:02
Mode 6	49.44	5.567	6.7	3.519		07-03-2007 17:52:21
Mode 7	71.41	4.745	4.06	4.057		07-03-2007 17:52:36
Mode 8	81.06	3.017	5.186	5.216		07-03-2007 17:52:40
Mode 9	89.83	12.59	6.286	5.104		07-03-2007 17:53:46
Mode 10	106.8	6.728	3.928	3.149		07-03-2007 17:54:08
Mode 11	143.5	38.38	4.811	4.174		07-03-2007 17:54:36

Peak Picking SV Plot for the Concrete Bridge



Mode	Frequency [Hz]	Std. Frequency...	Damping Rat...	Std. Damping ...	Comment	Creation Date ...
Mode 1	16.11	4.227	3.889	2.254		12-03-2007 02:04:56
Mode 2	19.3	3.365	4.016	3.542		12-03-2007 02:03:50
Mode 3	20.09	3.424	2.616	1.065		12-03-2007 02:04:37
Mode 4	21.84	2.636	2.183	926.6m		11-03-2007 17:05:37
Mode 5	37.09	3.127	1.684	476.6m		11-03-2007 17:05:39
Mode 6	33.28	6.383	2.438	2.384		11-03-2007 17:05:41
Mode 7	39.68	8.218	2.332	2.684		11-03-2007 17:05:43
Mode 8	44.33	8.763	3.736	2.798		12-03-2007 03:05:19
Mode 9	56.17	13.8	7.384	7.094		11-03-2007 17:07:41
Mode 10	69.04	24.63	11.79	9.514		11-03-2007 17:07:58
Mode 11	78.92	12.63	8.881	11.01		11-03-2007 17:08:01
Mode 12	88.42	7.207	8.988	11.86		11-03-2007 17:07:31