Structural Health Monitoring, History, Applications and Future. A Review Book

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Structural Health Monitoring

History, Applications and Future

A Review Book

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Open Science

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Summary

All structures, including critical civil infrastructure facilities like bridges and highways, deteriorate with time due to various reasons including fatigue failure caused by repetitive traffic loads, effects of environmental conditions, and extreme events such as an earthquake. This requires not just routine or critical-event based inspections (such as an earthquake), but rather a means of continuous monitoring of a structure to provide an assessment of changes as a function of time and an early warning of an unsafe condition using real-time data. Thus, the health monitoring of structures has been a hot research topic of structural engineering in recent years.

First, this work highlights the structural damage detection methods as a step of structural health monitoring (SHM) via discussing the differences between global and local damage detection techniques and the merits of each technique. Furthermore, it presents the interpretation of the localization of the derivatives of response-based damage detection methods.

Next, the state of the art of different global damage detection methods and their applications are presented. Thus, vibration-based methods, static response-based method, physical model-based method, as well as implementation of long-term SHM to critical civil structures are discussed.

Finally, the state of the art and the future of different types of response measurements and their applications in SHM of civil structures are reviewed. Also, some advances in measurement responses; such as laser doppler vibrometer (LDV) and fiber optic sensors (FOSs) are illustrated. General conclusions as well as recommendations for future research are presented.

Chapter 1

Overview and Scope

1.1 Introduction

All structures, including critical civil infrastructure facilities like bridges and highways, deteriorate with time. This deterioration is due to various reasons including fatigue failure caused by repetitive traffic loads, effects of environmental elements, and extreme events such as an earthquake. If the damages remain undetected, the structure may have a reduced margin of safety or have serviceability problem. Consequently, the integrity of the structural systems has big chances of a collapse, resulting in loss of life and property. Increasing concern about the status of existing structures, particularly after earthquakes, has motivated numerous studies on damage detection using various non-destructive evaluation methods.

Structural health monitoring (SHM) is the process of implementing a damage detection strategy. This process involves the observation of a structure over a period of time using periodically spaced measurements, the extraction of features from these measurements, and the analysis of these features to determine the current state of health of the structural system. The output of this process is periodically updated information regarding the ability of the structure to continue to perform its desired function in light of the inevitable aging and degradation resulting from the operational environments. Based on the monitored state, appropriate repair, rehabilitate, and/or strengthening of structures are decided to keep these structures operational and further to lengthen their lives. Because the cost for monitoring and repair is much lower than the cost for reconstruction of new structures, monitoring is vital for civil infrastructure facilities, which form the lifeline of our countries' economy. That is the reason that Japan, USA, China, New Zealand and other countries have been doing concerted efforts to instrument bridges, important buildings, and dams, Abe (1998).

Current damage detection methods are either visual inspection or localized experimental methods such as acoustic emission, ultrasonic methods, etc. All of these experimental techniques require that the vicinity of damage is already known and the portion of the structure being inspected is readily accessible. Related to these limitations, these methods can detect damage on or near the surface of the structure. In addition, the problem of current practice is also compounded by the shortage of experienced inspectors and the inevitable time delay caused by in-depth structural analysis. As a result, the need for additional global damage detection methods that can be applied to complex structures has arisen.

In view of the pre-mentioned limitations of visual and localized experimental methods of damage detection, a better approach is one that uses global indices. Global damage identification techniques can be classified as 'response-based method' or 'model-based method'. The response-based method depends only on experimental response data from structures; while the model-based method assumes that a detailed numerical model of the structure is available for damage identification, Fan (2011).

A typical procedure for response-based damage detection method involves the modal and/or static tests of the structural system from which the responses due to external excitations are measured. Since the dynamic and static responses of a structural system are functions of the structural parameters, these parameters may be identified by using the changes in dynamic and/or static characteristics. One of the consequences of the development of damage is the decrease in local stiffness, which in turn results in changes in some of the responses. It is therefore, necessary that the dynamic and/or static characteristics of the structure be monitored for damage detection and safety assessment. On the other hand, in model-based damage detection method, one makes use of general framework of finite element model refinement (FEMR), Zimmerman and Kaouk (1994). Thus, the damage detection problem is considered as a particular case of the general model-updating problem, where the aim of FEMR is to seek a refined model with its modal parameters in agreement with those obtained from the experiment.

Indeed, model-based methods are generally capable of estimating existence, location, and absolute severity of the damage. However, response-based methods are typically limited to the existence and location of the damage when applied to civil engineering structure, since an unsupervised learning mode has to be used, Guan and Karbhari (2008). It is worth to mention that it is not mandatory to use only one of the above methods in SHM. Some researchers have combined both modal and static responses for damage identification (Lee *et al.* (2010), Oh and Jung (1998), and Oh *et al.* (1999)). Other researchers have combined both modal response and model-based method for more effective SHM, Guan and Karbhari (2008).

Indeed, health monitoring has expanded because of the advances in data processing, computational power, and economy of microprocessors. Also, this expansion is attributed to the advances in mechanical static/dynamic measurements such as transducers, and introduction of LASER, (light amplification by stimulated emission of radiation), in sensing and gauging applications. LASER can be used directly as in scanning laser doppler vibrometer (SLDV), or indirectly as in the fiber optic sensors (FOSs), Abdo (2002).

1.2 Objectives of Study

According to the previous aspects, the main objective of this study is to review the state of the art as well as the future of the following:

- Local and global structural damage detection methods,

- Theoretical background of global damage detection methods,
- Global damage detection techniques and their applications, and
- Response measurements and their applications.

1.3 Scope of Study

The book contents are organized as follows:

Chapter 1: It introduces the book via situating the subject and clarifying the motivating objectives of this book. It also highlights the organization of the book.

Chapter 2: It highlights the structural damage detection methods as a step of SHM, and it discusses the difference between global and local damage detection techniques. Also, it presents the merits as well as the disadvantages of conventional methods for local damage detection techniques. Furthermore, it presents the interpretation of the localization of the derivatives of response-based damage detection methods.

Chapter 3: It presents the state of the art of different global damage detection methods. Thus, it contains: vibration-based methods, static response-based method, physical model-based method, as well as implementation and future of long-term SHM to civil structures.

Chapter 4: It highlights the state of the art and the future of different types of response measurements and their applications in SHM. Thus, some advances in measurement responses, such as laser Doppler Vibrometer (LDV) and fiber optic sensors (FOSs) are presented. It also presents the application of these sensors to civil engineering structures.

Chapter 5: It summarizes the general conclusions of this book. Also, it exhibits some suggestions and recommendations for future research related to structural health monitoring which remain unsolved yet.

- List of references is given in the bibliography at the end of this book.
- Appendices A and B introduce the algorithms used for damage localization using changes in dynamic and static responses, respectively.
- All symbols and notations are defined wherever they appear.

Chapter 2

Structural Health Monitoring

2.1 Introduction

Most civil engineering structures such as multistory buildings, towers, bridges, and offshore platforms accumulate damage gradually during their service life or suffer sudden damage due to natural disasters. From the viewpoint of serviceability and safety of structures, an important issue is the detection of structural damage, Koh *et al.* (1995). If the damages remain undetected, the structure may have a reduced margin of safety or have serviceability problem. Consequently, the integrity of the structural system has big chances of a collapse, resulting in the loss of human life and property. Increasing concern about the status of existing structures, particularly after earthquakes, has motivated numerous studies on damage detection using various non-destructive evaluation methods.

Briefly, Structural Health Monitoring (SHM) is a fairly new engineering field oriented to the development of damage detection systems able to facilitate the transition from scheduled maintenance to condition based maintenance, Semperlotti (2009). Engineers define health monitoring as the measurement of the operating and loading environment and the critical responses of a structure in order to track and evaluate the symptoms of operational anomalies and/or deterioration or damage that may impact service or safety reliability. Indeed, SHM resembles to a great extent the nervous system of humans. The nervous system of humans consists of a complex collection of nerves and specialized cells and the main processing unit (brain). The nerve cells transmit signals between different parts of the body and the brain. The brain is the main control unit for receiving and processing information as well as issuing instructions. In the same manner, SHM consists of a sensory network to gather information and a control unit for data processing and decision making. The similarity between the SHM and the nervous system of humans is illustrated in Fig. 2.1.

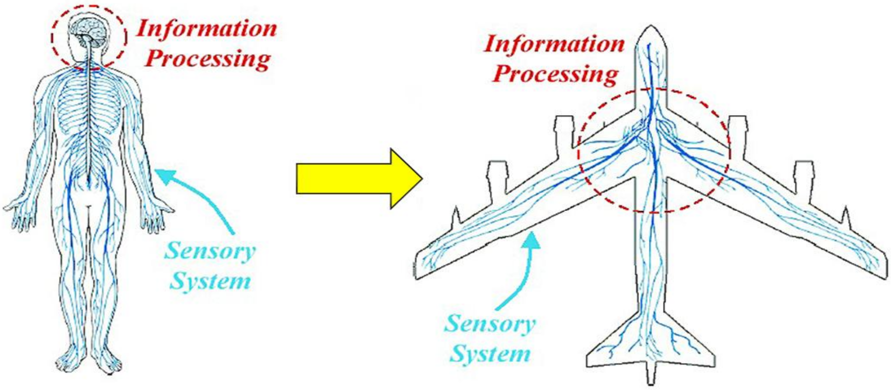

Figure 2.1 *Similarity between the SHM and the nervous system of humans.*

It is important to mention that besides damage detection, the objectives of structural monitoring systems may be extended to include: quality control during the construction, serving as a warning system under successive loading, as well as condition assessment about its serviceability and ultimate limit state. So, one of the important works to be done by civil engineers is to maximize the degree of mobility of the system. This requires not just routine or critical event (such as an earthquake) based inspections, but rather a means of continuous monitoring of a structure to provide an assessment of changes as a function of time and an early warning of an unsafe condition using real-time data. Based on the monitored state, appropriate repair and/or strengthening of structures are properly programmed to keep these structures operational and further to lengthen their lives. Therefore, health monitoring of civil structures has been attracting much attention from research community and practicing engineers, as more and more structures are suspected to deteriorate by aging, especially, in many development countries, Abe (1998).

Preventing the nation's infrastructure is dependent on the successful implementation of structural health monitoring concepts. According to Chang (2003), the engineering structural health concept encompasses four distinct subsets:

- Sensor allocation and measurements,
- Structural identification,
- Damage or degradation detection, and
- Decision making.

Indeed, a successful health project dealing with "structural health" in civil engineering should consider all these four subsets simultaneously. Each subset of SHM is a major topic by itself. There have been numerous activities in different subsets of the field. However, many concepts still need further study; and the integrated field as a whole has not been studied and understood in full detail.

Damage identification methods can also be classified as ‘model-based method’ or ‘response-based method’. The model-based method assumes that a detailed numerical model of the structure is available for damage identification; while the response-based method depends only on experimental response data from structures, Fan (2011).

Based on the amount of information provided regarding the damage state, these methods can be classified as providing 4 levels of damage detection. The four levels are (Rytter (1993) and Peeters *et al.* (2001)):

- Identify that damage has occurred,
- Identify that damage has occurred and determine the location of damage,
- Identify that damage has occurred, locate the damage and estimate its severity,
- Identify that damage has occurred, locate the damage, estimate its severity, and determine the remaining useful life of the structure.

Based on the damage identification results, estimation of present structural capacity and remaining life can be performed at each stage. Detailed method to evaluate the remaining capacity and life of the structure is not covered in this book, but can be found in Lee (2005). A brief idea about different damage detection techniques and the theoretical background of global damage detection methods are presented in the following sections.

2.2 Global Versus Local Damage Detection

Damage detection is a challenging problem that is under vigorous investigation by numerous research groups using a variety of analytical and experimental techniques. Damage detection techniques may be classified as global or local. Global methods attempt to simultaneously assess the condition of the whole structure, whereas local methods focus non-destructive evaluation (NDE) tools on specific structural components. Among the numerous considerations which influence the choice and effectiveness of a suitable health monitoring method are: (1) the level of damage and deterioration concern, (2) the types of sensors used, (3) the degree of measurement noise pollution, and (4) the degree of a priori information about the condition of the structure, etc., Nakamura *et al.* (1998). Periodic inspection of structures is essential in most cases. Structures are generally rated and monitored once a year or once in several years according to the importance and the age of the structure, Uomoto (2000-a).

2.3 Damage Detection Using Conventional Methods

Current damage detection methods are either visual inspection or localized experimental methods such as acoustic emission, ultrasonic methods, etc. All of these experimental techniques require that the vicinity of damage is already known and the portion of the structure being inspected is readily accessible. The experience of the responsible parties inevitably varies, making it difficult to maintain a level of consistency from one inspector to another. In addition, this method may not be applicable for inspecting defects that do not appear on or near the surface of the structure. For such defects, NDE using local experimental methods is applied.

Initially, non-destructive testing techniques were introduced in civil engineering during the 40's. The principal need of the engineers was the in-situ determination of the homogeneity and the compressive strength of fresh concrete to be able, for example, to remove the formworks. The majority of these techniques (rebound hammer, pull-out test, etc.) is standardized, ACI (1995), and is based on the measurement of the surface hardness of the concrete. With the progressive ageing of the structures, the needs of the engineers evolved to the search for tools allowing the estimation of the mechanical properties of old materials, as well as the detection and the characterization of hidden defects. This request was at the origin of the appearance of many investigation techniques in the construction industry, and this from the 70's. The available localized experimental methods being used include: acoustic emission or ultrasonic methods, electro-magnetic field methods, X-ray, radiographs, eddy current methods, and thermal field methods, Doebling *et al.* (1996). Many papers concerned with different non-destructive testing methods and their applications in civil engineering have been published by Uomoto (2000-b).

Since the reinforced concrete is a heterogeneous material, concrete structures have many problems more than metallic materials. Indeed, mechanical properties of various metals and metal alloys are generally reproducible, and the problems encountered are generally caused by cracks. It is thus not surprising that the non-destructive testing methods developed for metals are for the detection and sizing of cracks. Unlike metals, concrete is a composite material produced by the association of various ingredients (water, sand, cement, aggregates, chemical agents). It contains from the outset a great number of defects, in the shape of small cavities, pores and interstices, and is also a material whose mechanical properties are not rigorously reproducible, even under the best conditions. In addition, various degradations appear and develop during the working life of the concrete, leading to the loss of the structural capacity of this material.

Table 2.1 lists the applications of NDE for existing concrete structures. In this table, one may recognize that each method has its advantages and disadvantages; this is the reason why the inspector engineers combine two or more methods for structural evaluation. In addition, these NDE techniques have some limitations such as: (1) the quality of the process is often dependent on the inspection personnel experience and knowledge, (2) results from one (local) area of a structure does not necessarily represent conditions at another area, and (3) as a result, it would be necessary to make measurements at a large number of points so as to have a good representation of the global structural condition. These constraints imply that NDE techniques, which are commonly used for localized evaluation of large structures, fail when used for complete evaluation of the global and local performance of the structure. The need for additional global damage detection methods that can be applied to complex structures is the motivation to the development of methods that examine changes in the dynamic/static characteristics of structures via monitoring their dynamic and/or static responses. The response-based techniques have the potential to evaluate the whole structure due to its simple setup and potential automation of data acquisition, data processing, and defect detection, Aktan and Grimmelsman (1999).

Table 2.1 *Application of NDE for existing concrete structures, Uomoto (2000-a).*

Items	**Type of damage**	**Method(s) to detect damage**	**Comments**
Appearance	Defects (surface)	Digital still camera, Thermograph	Including honeycomb, cold joints
	Defects (inside)	Sonic, Thermograph, Radar Ultra-sonic, X-ray, Impact echo	Voids inside and at the back of the structure
Strength and stiffness	Concrete strength	Rebound hammer	Problem in accuracy
	Modulus of elasticity	Ultra-sonic velocity	- - - - - - - - - -
Cracks and spalling	Distribution	Digital still camera, Thermograph	- - - - - - - - - -
	Crack width	Digital still camera, Thermograph	Direct measurement
	Crack depth	Ultra sonic	Effects of bars
	Cracking	Acoustic emission	Continuous measurements
Steel corrosion	Location	Natural potential	Location at that time
	Corrosion degree	Natural potential, Electric-current analysis	Periodic measurements required

2.4 Theoretical Background of Global Damage Detection

Global damage identification methods can be classified as ‘model-based method’ or ‘response-based method’. The model-based method assumes that a detailed numerical model of the structure is available for damage identification; while the response-based method depends only on experimental response data from structures, Fan (2011).

A typical procedure for response-based damage detection method involves the modal and/or static test of the structural system from which the responses due to external excitations are measured. Since the dynamic and/or static responses of a structural system are functions of the structural parameters, these parameters may be identified by using the changes in dynamic and/or static characteristics. One of the consequences of the development of damage is the decrease in local stiffness, which in turn results in changes in some of the responses. It is therefore, necessary that the responses of the structure be monitored for damage detection and safety assessment, Abe (1998). In fact, understanding of the relationships between the damage and the corresponding changes in the dynamic properties is the key point to detect damage in a structure.

2.4.1 Changes in Vibration Characteristics

Of the various structural health monitoring (SHM) techniques proposed to date, vibration-based SHM has drawn significant attention. The basic premise of vibration-based SHM is that changes in structural characteristics, such as mass, stiffness and damping, will affect the global vibrational response of the structure. The global nature of the vibrational characteristics of interest provides advantages compared with other health monitoring techniques, Guan and Karbhari (2008). The theoretical background of using changes in dynamic characteristics in damage detection is given by Abdo (2002). In the absence of any external force (free vibration), the equation of motion of an n-DOF structural dynamic system can be expressed as:

$$M\ddot{u} + C\dot{u} + Ku = 0 \tag{2.1}$$

in which *M*, *C*, and *K* are the global mass, damping, and stiffness matrices of the structure, respectively. The dots over the *u* denote the first and second derivatives of the displacement with respect to time. An eigenvalue problem of finding the natural frequencies and the displacement mode shapes for undamped systems is:

$$(K - \lambda M)\phi = 0, \tag{2.2}$$

where λ and ϕ are the eigenvalue and displacement eigenvector of the intact structure, respectively.

Considering a small perturbation, ΔK, in the stiffness matrix with similarly denoted changes in other parameters, Eq. (2.2) then becomes:

$$\{(K+\Delta K)-(\lambda+\Delta\lambda)(M+\Delta M)\}(\phi+\Delta\phi)=0. \quad (2.3)$$

Multiplying out and neglecting second-order terms, Eq. (2.3) becomes:

$$K\phi-\lambda M\phi-\lambda\Delta M\phi+\Delta K\phi-\Delta\lambda M\phi+K\Delta\phi-\lambda M\Delta\phi=0. \quad (2.4)$$

In actual structures, damage may often affect the stiffness matrix but not the mass matrix of the system, Shi and Law (1998). With $(K-\lambda M)\phi=0$ and ΔM=0, Eq. (2.4) leads to the following form:

$$\Delta K\phi-\Delta\lambda M\phi+K\Delta\phi-\lambda M\Delta\phi=0. \quad (2.5)$$

Multiplying through by ϕ^T gives:

$$\phi^T\Delta K\phi-\Delta\lambda\phi^T M\phi+(\phi^T K-\lambda\phi^T M)\Delta\phi=0. \quad (2.6)$$

Assuming that both K and M are symmetric matrices, which is consistent with Zimmerman and Kaouk (1994), the transpose of Eq. (2.2) is:

$$\phi^T(K-\lambda M)=0. \quad (2.7)$$

Post multiplying with $\Delta\phi$ gives

$$(\phi^T K-\lambda\phi^T M)\Delta\phi=0. \quad (2.8)$$

Hence, Eq. (2.6) reduces to be:

$$\phi^T\Delta K\phi-\Delta\lambda\phi^T M\phi=0, \quad (2.9)$$

or

$$\Delta\lambda = \frac{\phi^T \Delta K \phi}{\phi^T M \phi}, \tag{2.10}$$

which relates the changes in the eigen characteristics with the changes in the stiffness and mass of the system.

Considering the orthogonal condition $\phi^T M\phi = 1$, Eq. (2.10) becomes

$$\Delta\lambda = \phi^T \Delta K \phi. \tag{2.11}$$

It is obvious from Eq. (2.11) that the changes in eigenvalues (natural frequencies) and eigenvectors (mode shapes) are directly related to the stiffness of the structure. Therefore, any reduction in the stiffness will result in a drop in natural frequencies and/or modification in mode shapes. Thus, some researchers have used the changes in the natural frequencies and mode shapes in structural damage detection. However, they found that these changes are global and don't give any idea about the location of damage. On the other hand, the derivatives of mode shapes (slope, curvature, and higher derivatives) and strain mode shapes are found to be localized at the damaged region and have shown promise in structural health monitoring. More details about application of derivatives of mode shapes are found in Chapter 3.

2.4.2 Changes in Static Responses

Indeed, among different approaches developed for response-based structural damage detection, changes in displacement curvatures have shown promise for locating structural damage (Abdo (2012) and Chen *et al*. (2005)). Based on the mechanics of materials theory, curvature= $1/R$ (R is the radius of curvature) and deflection v are related by Eq. (2.12):

$$\frac{1}{R}=\frac{\dfrac{\partial^2 v}{dx^2}}{\left[1+\left(\dfrac{\partial v}{dx}\right)^2\right]^{3/2}}=\frac{M}{EI}, \tag{2.12}$$

where M is the bending moment, E is the Young's modulus and I is the moment of inertia of the cross section. Neglecting the second order of slope, curvature can be approximated by (2.13):

$$Curvature \approx v'' = \frac{\partial^2 v}{dx^2}. \tag{2.13}$$

Then, we can get the relationship between curvature, bending moment and stiffness, as follows:

$$\frac{\partial^2 v}{dx^2}=\frac{M}{EI}. \tag{2.14}$$

Equation (2.14) shows that curvature is a function of stiffness. Any change in stiffness due to any damage at a section should be evidenced by a change in curvature at that location.

To study the changes of displacement curvatures due to presence of damage, Abdo (2012) considered a beam element with length L. The beam has prismatic cross section and has stiffness K=EI with E and I as defined above. A very narrow zone of damage is assumed at $x_d - h/2 < x < x_d + h/2$ with the stiffness ($K - \Delta K$), where h/L << 1. As h goes to zero, the associated displacement, slope, bending moment and shear force approach those of the intact beam. However, if there is a small damage with length h, the curvature at the intersection suffers a jump; for instance, at $x_d - h/2$, the curvature satisfies $M = K\kappa^l = (K - \Delta K)\kappa^r$, where κ and M stand for the curvature and bending moment. Here, superscript l or r indicates the left or the right of the intersection, respectively. The amount of curvature discontinuity, $\varDelta\kappa$, is easily evaluated as follows:

$$\varDelta\kappa = \frac{M \;\; \varDelta K}{K\left(K - \varDelta K\right)}. \tag{2.15}$$

Since h is very small, this discontinuity leads to a spike in the displacement curvature; its height is related to ΔK since M is constant at the same section. If M is small, however, such a localized spike does not appear. The above example of the beam element, Abdo (2012), shows that the displacement curvature is a good damage indicator in detecting the location of damage(s).

Chapter 3

Global Damage Detection Methods and Their Applications

3.1 Introduction

In view of the pre-mentioned limitations of visual and localized experimental methods of damage detection in Section 2.3, a better approach is one that uses global indices. Global damage identification techniques can be classified as ‘response-based method’ or ‘model-based method’. The response-based method depends only on experimental response data from structures; while the model-based method assumes that a detailed numerical model of the structure is available for damage identification, Fan (2011).

A typical procedure for response-based damage detection method involves the modal and/or static tests of the structural system from which the responses due to external excitations are measured. Since the dynamic and static responses of a structural system are functions of the structural parameters, these parameters may be identified by using the changes in dynamic and/or static characteristics. One of the consequences of the development of damage is the decrease in local stiffness, which in turn results in changes in some of the responses. It is therefore, necessary that the dynamic and/or static characteristics of the structure be monitored for damage detection and safety assessment. On the other hand, in model-based damage detection method, one makes use of general framework of finite element model refinement (FEMR), Zimmerman and Kaouk (1994). Thus, the damage detection problem is considered as a particular case of the general model-updating problem, where the aim of FEMR is to seek a refined model with its modal parameters in agreement with those obtained from the experiment.

Indeed, model-based methods are generally capable of estimating existence, location, and absolute severity of the damage. However, response-based methods are typically limited to the existence and location of the damage when applied to civil engineering structure, since an unsupervised learning mode has to be used, Guan and Karbhari (2008).

It is worth to mention that it is not mandatory to use only one of the above methods in structural health monitoring. Some researchers have combined both modal and static responses for damage identification (Lee *et al.* (2010), Oh and Jung (1998), and Oh *et al.* (1999)). Other researchers have combined both modal response and model-based method for more effective structural health monitoring, Guan and Karbhari (2008).

3.2 Vibration-Based Methods

Over the past 30 years, detecting damage in a structure from changes in global dynamic parameters has received considerable attention from the civil, aerospace, and mechanical engineering communities, Cornwell *et al.* (1998). Aeronautical structures such as air planes and space crafts are more critical than civil structures due to the relatively small factor of safety (as less as 1.25) in the design of former ones. In addition, they are very expensive and operate in uncertain and severe environments (engine vibration, lightning strikes, corrosion, etc.) So, health monitoring is vital for the former structures. Indeed, vibration monitoring has already proved to be effective in the field of mechanical and aeronautical engineering to evaluate structural integrity using measured vibration response, Alampalli *et al.* (1992). The ratio of life-cycle-cost versus operational capability of aircraft structures has been significantly reduced over the past decades. One of the main reasons is the introduction of improved non-destructive evaluation (NDE) techniques which allows switching from safe-life to fail-safe design, Boller and Biemans (1997).

The basic principle of using vibration monitoring to assess structural integrity relies on the fact that dynamic response is a sensitive indication of the physical integrity of any structure. As a result of damage (or structural distress), local or global, there would be a reduction in stiffness and a change in the energy stored in the body, DiPasquale *et al.* (1990). The dynamic response of the linear structure under forcing function $F(t)$ is governed by the structural physical properties (i.e., stiffness, mass, and damping) as follows:

$$M\ddot{u} + C\dot{u} + Ku = F(t), \tag{3.1}$$

in which M, C, and K are the global mass, damping, and stiffness matrices of the structure, respectively and the dots over the u denote the first and second derivatives of the displacement with respect to time. Using Fourier transform, Eq. (3.1) can be rewritten in frequency domain as:

$$S(\omega)\, x(\omega) = F(\omega), \tag{3.2}$$

where $S(\omega) = (-\omega^2 M + i\omega C + K)$ is called the system matrix (dynamic stiffness matrix) and is dependent on frequency (ω), and $x(\omega)$ and $F(\omega)$ are simply the vectors of nodal degrees of freedom and nodal forces, respectively. Therefore, any changes in the structural physical properties will in turn, lead to changes in the vibrational response as characterized by the modal characteristics (natural frequencies, mode shapes, and modal damping values).

Due to the fact that each vibration mode has a different energy distribution, any localized damage will affect each mode differently depending on the nature, location, and severity of damage. Modal parameters are also sensitive to boundary conditions (physical constraints) of the structure. The changes in the vibration characteristics can then be used as indicators of damage detection. Techniques of detecting damage in a structure by monitoring these changes have attracted much attention in recent years, and many methods have been proposed.

Vibration testing to measure the response of the structure is usually done either by artificially induced excitation forces or ambient forces (wind, traffic loading, etc.) in the service environment. Measurements of the structural response are usually achieved by accelerometers, or other response transducers, as will be discussed in the next chapter. In forced-vibration testing, the frequency response function (FRF) is computed by dividing the discrete Fourier transform (DFT) of the output response by that of the load input. Details of FRF computation can be found in Ewins (2000). Forced-vibration testing is preferred to ambient testing because it produces data from which system parameters can be better identified, Salawu and Williams (1995). However, for large civil structures, ambient sources of vibration are not only cheap but also more representative of the true excitation to which the structure is subjected during its lifetime, Peeters *et al.* (2001). Indeed, there is no alternative to ambient sources for continuous health monitoring.

In fact, the modal approach can be considered as the main stimulus for the growth of the field of vibration-based structural health monitoring and damage detection. The attractiveness of this approach can be attributed to the fact that dynamic characterization of the structure is in many cases easier to perform in the field than static characterization. Due to the advances in sensor technology, low input energy levels are usually sufficient to produce sets of measurable dynamic response. Hence ambient sources can be used as the excitation for structures eliminating the need for expensive excitation devices. Relatively accurate results of natural frequencies, mode shapes, and modal damping can be extracted from vibration based measurements due to advances in response measurements, Guan and Karbhari (2008).

Detecting damage throughout the changes in dynamic characteristics has many advantageous features. Among those advantages are:

1. Damage can be located and sized without solving a system of equations,
2. Rely only on the measured data without any prior theoretical model,
3. Only lower mode shapes are needed for the analysis, and
4. Damage can be located and sized using few modes.

Furthermore, this kind of health monitoring technique in which a priori information about the nature of the model is not needed, has a significant advantage when dealing with real-world situations. In the next subsections, a brief review of the dynamic characteristics used in global damage detection techniques is presented.

3.2.1 Natural Frequencies

The observation that changes in structural properties cause changes in vibration frequencies was the point of departure for using modal methods for damage identification and health monitoring. Numerous studies have indicated that any structural damage reflects a decrease in natural frequencies of the structure. To realize this, an eigenvalue problem of finding the natural frequencies and the displacement mode shapes for undamped system is:

$$\left(K+\lambda_i M\right)\varphi_i=0\,, \tag{3.3}$$

in which K and M are as defined above, the stiffness and mass matrices, and λ_i and φ_i are the i th eigenvalue and displacement eigenvector of the intact structure, respectively. However, when K is changed, Eq. (3.3) becomes as follows:

$$\left(K'+\lambda_i' M\right)\varphi_i'=0, \tag{3.4}$$

in which K' is the stiffness matrix of the damaged structure, λ_i' and φ_i' are the i

th eigenvalue and displacement eigenvector of the damaged structure, respectively. It is obvious that any reduction in the stiffness of the structure will decrease its natural frequencies.

The most useful damage detection methods (based on dynamic testing) are probably those using changes in resonant frequencies because frequency measurements can be quickly conducted and are often reliable. Another advantage is the global nature that allows the measurement points to be chosen to suit the test situation. The use of the natural frequency of a structure to determine global defects and deterioration was investigated extensively. Salawu (1997) gave a literature review of the state of the art of damage detection using changes in natural frequencies.

Early research relating to global damage detection was focused on the information of natural frequency changes (Adams *et al.* (1978), Gudmundson (1982), Mazurek and DeWolf (1990), Salane and Baldwin (1990), Ventura and Adebar (1997)). This focus was extended to define the changes in modal frequencies as functions of damage characteristics. Cawley and Adams (1979) gave a formulation to detect damage from frequency shifts using sensitivity analysis. The technique was applicable to all systems that were amenable to finite element analysis. However, the formulation did not account for multiple-damage locations. Other damage location method using frequency changes was proposed by Hearn and Testa (1991). They perturbed the stiffness matrix of a finite element model and compared the resulting changes in natural frequencies. Richardson and Mannan (1992) demonstrated that a fault (a small saw cut in an aluminum plate) can be remotely detected and located by using a set of modal data from the undamaged structure, along with the frequencies of its modes after inflicting damage. Messina *et al.* (1998) introduced a new correlation coefficient to locate as well as size the damage which required information about the changes in only a few of natural frequencies between the undamaged and damaged states. Fayyadh and Abdul Razak (2012) conducted a vibration testing to detect the deterioration of the elastic bearing supports of a simply supported reinforced concrete bridge girder. They found that natural bending frequencies were sufficiently sensitive to detect the deterioration in the elastic bearing supports and the trends in the shifts were consistent. This is applicable to all the modes considered in study except the third mode which showed an abnormal trend.

Actually, the changes in frequency imply the presence of crack or damage in a structure. However, determining the location of the crack, knowing only the changes in the frequencies, is a completely different question. This is because, e.g., cracks at two different locations associated with certain crack length may cause the same amount of frequency change. It is of interest to mention that at the modal nodes or modal lines (points or lines of zero modal displacements) the stress has a minimum value for the particular mode of vibration. Hence the minimal change in a particular modal frequency could mean that the defect may be close to the modal node/line.

Abdo (2002) carried out a numerical study to investigate the effect of damage on natural frequency of a plate-like structure. He concluded that the changes in frequency imply the presence of damage but the determination of the damage location, knowing only the changes in frequencies, is a completely different question. Furthermore, the natural frequencies of the lower modes do not give an indication for a very small amount of damage. Also, Abdo (2003) carried out experimental tests on a large scale multi-storey reinforced concrete building on a large shaking table. It is worth to mention that the experimental results emphasize the numerical results. Recently, Fan (2011) has shown that the modal frequency shift alone may not be reliable for damage identification of fiber reinforced polymer sandwich beams/panels. Thus, we are in need to other characteristics to pinpoint damage location.

3.2.2 Modal Damping

It is generally not possible to predict the damping in each mode of vibration from a theoretical model and so there is nothing with which to compare measurements of modal damping from the tests, Ewins (2000). However, such information is useful as it can be incorporated into the theoretical model, albeit as an approximation, prior to that being called upon to predict specific response levels. Indeed, in most numerical simulation the effect of damping is neglected, Abdo (2002).

As cracks create new surfaces, damping ratio will increase when damage progresses in the structure. The effect of debonding on the modal damping of a sandwich panel was investigated by Peroni *et al.* (1991), who showed that for some modes, damage caused a slight increase in damping coefficient. Also, Ratcliffe (1997) found a small increase in damping for the first five modes of a steel plate. Recently, Prasad and Seshu (2010) have performed an experimental modal analysis on reinforced concrete beams subjected to cyclic loading. They observed that the trend for the damping ratios obtained increase as the level of damage increases. Also, they observed that the values of damping ratio decrease with higher modes indicating that lower modes have the tendency to decay much earlier. However, for large structures, damping ratios have shown large variance, inconsistency, and have not followed a particular trend, Mazurek and DeWolf (1990). Salane and Baldwin (1990) agreed with Salawu and Williams (1994) that variations in viscous damping ratios could not be predictably related to the deterioration of the bridges. The large scatter usually observed (Alampalli *et al.* (1992), Christensen *et al.* (2001), and Salawu and Williams (1995)) in estimated damping values excludes itself from being used as a means of integrity assessment.

3.2.3 Mode Shapes, MAC, and COMAC

The methods using the natural frequencies were confined in their applications to truss structures used in space structures, oil platforms, and other global structure responses. Yuen ((1985), (1986)) showed for a beam with different end conditions that there is a systematic change in the first mode shape with respect to the damage location. He simulated the damage as a reduction in the stiffness of each structural element and used the finite element analysis to obtain the natural frequencies and the mode shapes. The author mentioned the need for some ortho-normalization process (i.e., $\varphi_i^T M \varphi_i = 1$), where M is the mass matrix and φ_i is the mode shape. Rizos *et al.* (1990) developed an analytical model for vibration of a beam with an open crack. This method is applicable for all one-dimensional structures and its accuracy would be progressively smaller with more complex structures. Wolff and Richardson (1989) investigated an aluminum flat plate before and after damage. They found large differences in mode shapes due to damage but these changes did not pinpoint the location of the fault. They also observed that the movements of the node lines, nodes of zero displacement mode shape, clearly reflected the existence of damage. Having investigated both aluminum and steel plates with holes, Richardson and Mannan (1993) stated that the minimum requirement to locate and quantify a fault is that the mode shapes of all the dominant modes of undamaged structure must be known. Wong *et al.* (2001) observed that for a circular plate, the location of damage could be detected using the residual mode shape. Their experimental tests showed good agreement with the numerical results.

Tang and Leu (1991) found that changes in mode shapes of the structure were more sensitive indictors for locating structural damage than natural frequencies.

Alampalli *et al.* (1992) found that natural frequencies and mode shapes were able to indicate the existence of simulated and real damage in model bridges. Fox (1992) illustrated that graphical comparisons of relative changes in mode shapes proved to be the best way of detecting the damage location when only resonant frequencies and mode shapes were examined. Salawu and Williams (1994) demonstrated that the relative difference of the mode shapes did not typically give a good indication of damage using experimental data. Actually, they pointed out that the most important factor was the selection of the modes used in the analysis. Chen *et al.* (1995) evaluated the dynamic response of two channel steel-beams, which have similar frequency response observed on actual bridges. The results indicated that the measured free-vibration mode shapes could not identify the damage location. Via a numerical study on a plate-like structure, Abdo (2002) confirmed the fact that the changes in displacement mode shapes did not pinpoint the damage location. This is the motivation to examine the other characteristics of the mode shapes to pinpoint the structural damage.

In engineering analysis and design, it is often desirable to compare and correlate results from a theoretical analysis with measured data, when available, in order to validate the theoretical model. The field of experimental and analytical modal analysis has a number of methods that can be used for such correlation studies, Ewins (2000). The methods available for comparing theoretical and measured vibration data are applicable to any two sets of data: measured/measured, theoretical/theoretical, and theoretical/ measured. In addition many researchers have used them as guidance for damage detection by using mode shapes before and after damage.

The two most commonly used methods to compare two sets of vibration mode shapes before and after damage are modal assurance criterion (MAC) and co-ordinate modal assurance criterion (COMAC), Salawu and Williams (1995).

The MAC between the q th mode of the first data set A and the r th mode of the second data set B is defined (Allemang and Brown (1982)) as follows:

$$MAC(\{\phi_A\}_q, \{\phi_B\}_r) = \frac{\left| \{\phi_A\}_q^T \{\phi_B\}_r \right|^2}{(\{\phi_A\}_q^T \{\phi_A\}_q)(\{\phi_B\}_r^T \{\phi_B\}_r)}, \tag{3.5}$$

in which $\{\varphi_A\}_q$ is the mode shape vector for mode q of data set A, and $\{\varphi_B\}_r$ is the mode shape vector for mode r of data set B. A MAC value close to one (more than 0.9) indicates that the two modes are well correlated, and a value close to zero (less than 0.05) is indicative of uncorrelated modes.

The other famous method to compare and correlate two sets (A and B) of vibration mode shapes is COMAC. If i is the measurement location, N is the total number of correlated mode pairs, and $({}_i\,\varphi_A)_j$ is an element of mode shape vector for set A in correlated mode pair j, then the COMAC for measurement location i is defined (Lieven and Ewins (1988)) as follows:

$$COMAC(i) = \frac{\left[\sum_{j=1}^{N} ({}_i\phi_A)_j ({}_i\phi_B)_j \right]^2}{\sum_{j=1}^{N} ({}_i\phi_A)_j^2 \sum_{j=1}^{N} ({}_i\phi_B)_j^2}. \tag{3.6}$$

A COMAC value close to one indicates good correlation, at the selected location, between the two data sets and a value close to zero is indicative of uncorrelated data sets. In Eqs. (3.5) and (3.6), the modes have been assumed to be real.

Alampalli *et al.* (1992) concluded that mode shapes and MAC values could be used to further identify damage locations when frequency changes had been observed. Fox (1992), found that MAC was reasonable indicator of damage but a little insensitive, particularly in the presence of experimental scatter. MAC values were found to indicate which modes are being affected by most by damage, Salawu and Williams (1995). Fayyadh and Abdul Razak (2012) found that MAC index is a good indicator to classify the deterioration reason, where for the elastic bearing deterioration the first mode has higher change in MAC values, and higher modes have very small change. On the contrary, for the structural element defect the higher modes have higher change in MAC values, and first mode has very small change. COMAC values were found to be a useful parameter to detect errors in spatial model by Kim *et al.* (1992). Salawu and Williams (1994) showed that both MAC and COMAC were sensitive to damage but unable to clearly indicate the damage location. However, Pandey *et al.* (1991) found that both MAC and COMAC could not detect the damage in its earlier stages.

Having investigated a numerical study on a plate-like structure, Abdo (2002) found that MAC and COMAC values can be used to compare different mode shapes in order to distinguish identical and dissimilar modes. However, it is shown that MAC values are not sensitive to the damage, at least for all of the studied scenarios of damage of the plate models. This is because in calculating MAC values, differences are averaged over all the measurement points. On the other hand, COMAC values are found to be a useful parameter as guidance for damage location for severe damage although they are inconclusive for earlier stages of damage. Furthermore, COMAC values have problems in identifying multiple damage locations with different degrees of severity.

In view of the shortcomings of the damage detection methods using natural frequencies and/or mode shapes, researchers start to seek other features that are more sensitive to damage. Derivatives of mode shapes are found to be good candidates.

3.2.4 Derivatives of Mode Shapes

(1) Rotations of Mode Shapes

Yuen (1985) showed for a cantilever beam that the changes (before and after damage) in rotation eigenparameter (normalized by the natural frequency) take a step jump in value when crossing over the damaged zone. Abdo and Hori (2002) illustrated with theoretical analysis that the first derivative of mode shapes is a good indicator of damage. The numerical results of a steel plate model demonstrate the usefulness of the changes in the rotation of mode shape as a diagnostic parameter in detecting and locating damage in the connection(s) of the steel plate with different boundary conditions. It is shown that changes in the derivative (rotation or slope) of the mode shapes are more sensitive than the changes in the displacement mode shapes. Also, they found that the rotation of mode is localized in the region of damage for initiation or extension of damage. The robustness in using changes in the above index is that it does not need higher modes of the structure to be used. Furthermore, this method has the ability to pinpoint a small amount of damage. Thus, the method is promising if the rotation of mode shapes could be measured.

(2) Curvatures of Mode Shapes

Pandey *et al.* (1991) demonstrated for cantilever and simply supported beams that absolute changes in mode shape curvature could be a good indicator of damage for the beam structure using finite element method. If u_i is the displacement mode shape value at a measurement site i, then u_{i+1} and u_{i-1} can be expressed in terms of u_i using a Taylor series expansion as:

$$u_{i+1} = u_i + u_i'(L) + \frac{u_i''}{2!}(L)^2 + \dots \tag{3.7}$$

$$u_{i-1} = u_i + u_i'(-L) + \frac{u_i''}{2!}(-L)^2 + \ldots \tag{3.8}$$

When we add Eq. (3.7) to Eq. (3.8) and reorganize, the modal curvature value at the *i* th node is expressed as follows:

$$u_i'' = \frac{u_{i+1} - 2u_i + u_{i-1}}{L^2} + O(L^2), \tag{3.9}$$

where u_i, u_{i-1}, and u_{i+1} are the current, previous, and next displacement mode shape values, and L is the length of the element. It should be noted that the spacing between measurement sites must remain constant in order for Eq. (3.9) to be valid. Indeed, Eq. (3.9) is called the second central finite divided difference, or in short, central difference. It is apparent that Eq. (3.9) is an approximation due to the truncation error term $O(L^2)$.

Quan and Weiguo (1998) showed that for the steel deck of a bridge, the curvature of mode shapes are the best among three damage recognition indices based on mode shapes, (the COMAC, the flexibility, and the curvature of mode shape). In addition, they found that some first vibration mode shapes, whether vertical or horizontal modes, could be used equally to detect damage in the steel deck. Ratcliffe (1997) successfully used a finite difference Laplacian function, which represents the curvature of the mode shape, to identify the location of damage as little as about 10% in a uniform beam. He stated that this method had a problem for less severe damage because it needed further processing and gave smearing location of damage. From the displacements of mode shapes of a reinforced concrete beam, Maeck and De Roeck (1999-b) developed a technique to calculate the modal curvatures and torsion rates without numerical derivation. The drawback of the objective function they used was the difficulty in choosing appropriate factors. Abdel-Wahab and De Roeck (1999) investigated damage detection in bridges using modal curvatures. They found that modal curvatures of the lower modes were in general more accurate than those of higher ones. They suggested that the difference in the modal curvature to be averaged over all modes, when more than one fault exists in the structure. Indeed, the use of all modes might mask the indication of damage due to insensitive modes to a certain amount of damage. Applications of modal curvature to concrete structures were found to be promising, Maeck and De Roeck (1999-a), and by Pirner and Urushadze (2001).

Abdo (2002) extended the application of curvature of mode shapes to plate-like structures. The author used the invariant of curvature of mode shapes (*IC*) as follows:

$$IC = \frac{\partial^2 u_z}{\partial x^2} + \frac{\partial^2 u_z}{\partial y^2}, \tag{3.10}$$

where the two terms of the right side is the curvature in *x*- and *y*-directions, respectively. Via comparing the modal curvatures before and after damage, the author confirmed the fact that the curvature of mode shapes is localized at the damage location and that it is promising as damage indicator even for small amount of damage. However, this method requires dense mesh of measurements and it is sensitive to the existence of noise due to approximate estimation of curvature from the displacement mode shapes, as shown in Eq. (3.9). The author states that the method will give more reliable results if the curvatures of mode shapes are measured directly without approximation.

Abdo (2004) carried out a comparative study between curvature of mode shapes and uniform load surface curvature techniques used in damage identification. The numerical results show that both of the two methods can accurately locate single damage with different damage characteristics (location and severity). Also, curvature of mode shapes does not need more than one of the lower mode shapes for damage detection. However, the two methods have shown less sensitivity to specific types of damage when applied to multiple damage locations. The damage localization algorithm and some results of using curvatures of mode shapes in damage detection are found in Appendix-A. Abdo and Abdou (2005) have investigated the robustness of using curvatures of high-order mode shapes (which can be measured by advanced sensors) in damage identification. They found that both low- and high-order mode shapes gave successful results even though the low-order mode shapes gave smoother localization than that of the high-order mode shapes.

Sazonov and Klinkhachorn (2005) demonstrated that the maximum error bound of Eq. (3.9) considering both truncation error and measurement error in *u* can be expressed as:

$$\left|E[u_i'']\right| \le \frac{\varepsilon\left(\left|u_{i+1}\right| + 2\left|u_i\right| + \left|u_{i-1}\right|\right)}{L^2} + \frac{M_4}{12}L^2, \tag{3.11}$$

where $\left|E[u_i'']\right|$ is the modal curvature error bound, ε is the maximum relative random multiplicative error of mode shape u, and M_4 is a constant term determined by the maximum value of the $4th$ derivative of u. The first term on the right hand side of Eq. (3.11) corresponds to the noise in mode shape data. The second term corresponds to the truncation errors. When the spacing between measurement sites, L, is relatively large, the second term tends to dominate Eq. (3.11). With a reduction in spacing L, the first term tends to grow larger and gradually become the dominant factor in the error. Thus it appears that, contrary to common belief, the results of damage detection method may not be able to benefit from high-spatial resolution measurements if it depends on modal curvature computed using a numerical differentiation procedure. So, we are in need to further improvement in the estimation of curvature of mode shapes.

The accuracy of Eq. (3.9) can be further improved following Chapra and Canale (2001) by including additional terms in the Taylor series expansion, leading to an expression where the truncation error is of order L^4 as follows:

$$u_i'' = \frac{-u_{i+2} + 16u_{i+1} - 30u_i + 16u_{i-1} - u_{i-2}}{12L^2} + O(L^4). \tag{3.12}$$

In damage identification of a two-span aluminum beam, Guan and Karbhari (2008) found that for relatively dense measurement sites, the difference between Eq. (3.9) and Eq. (3.12) is insignificant. However, Eq. (3.12) performs significantly better than Eq. (3.9) and it is recommended to be used for less measurement sites.

(3) Fourth Derivatives of Mode Shapes

Whalen *et al.* (2006) investigated the fourth derivative of mode shapes in detecting damage and its application to a full scale plate-girder bridge. They found that the method showed excellent accuracy in locating stiffness reductions in beam-like structures while using a small number of measured modal displacement points. Ismail *et al.* (2006) used mode shape fourth derivative divided by the mode shape displacement data (local stiffness indicator) to detect damage in reinforced concrete (RC) beams. They found that the local stiffness indicator is good at locating damage in RC beams but it produced poor results in the vicinity of supports. Whalen (2008) investigated the behavior of high-order mode shape derivatives in damaged, beam-like structures. All high-order modal derivative discontinuities display strong localization under the assumption of beam-like vibrations.

Abdo (2012) extended the application of using high-order mode shape derivatives, especially the fourth derivative, in damage detection of plate-like structures. It is shown that the fourth derivative (curvature of curvature) of mode shape is promising in detecting and locating structural damage in plate-like structures, since it is localized at the damage location(s) even for a small amount of damage; using only one of the mode shapes. Both low- and high-order mode shapes give successful results. Also, damage indices of the fourth derivatives give smoother localization and consequently better damage identification than those of curvature of mode shapes. Furthermore, using high-order modes (which can be measured by advanced sensors) does not improve the results of damage identification using the fourth derivative. Unfortunately, damage detection using changes in fourth derivative of mode shapes is more sensitive to measurement noise.

3.2.5 Strain of Mode Shapes

Many researchers have demonstrated the feasibility and usefulness of measuring strain of mode shape with the help of strain gauges instead of measuring acceleration and displacement mode shapes, Bernasconi and Ewins (1989), Li *et al.* (1989), and Tsang (1990). Having noticed that curvature and bending strain for beams are directly proportional, many researchers investigated strain of mode shape as a damage indicator. Chance *et al.* (1994) found that numerically calculated curvature from mode shapes resulted in unacceptable errors. They noticed that using the measured strains improved the results drastically. Yao *et al.* (1992) concluded that measured strains of mode shapes were more effective at identifying the location of damage than displacement mode shapes for damage in a steel truss. Chen and Swamidas (1994), Dong *et al.* (1994), Yam *et al.* (1996), and Kim and Kwak (2001), found that the changes in strain of mode shapes facilitated the location of damage. Abdo (2002) has done a parametric study on using strain of mode shapes as damage indicator in plate-like structures. The results clarify that the strain of mode shapes is a good candidate for structural damage identification. It is found to be effective and robust in detecting and locating both initiation and extension of damage in a structure. Multiple damage locations with different levels of severity can be fairly located using the damage localization method.

3.2.6 Strain Energy Method

Shi and Law (1998) introduced a damage localization method using modal strain energy change (MSEC). They defined the modal strain energy of the jth element and the ith mode before and after the occurrence of damage as:

$$(MSE_{ij})_u = \phi_{ui}^T K_j \phi_{ui}\text{, and } (MSE_{ij})_d = \phi_{di}^T K_j \phi_{di}, \tag{3.13}$$

where subscripts u and d denote undamaged and damaged cases. They used the original stiffness matrix for both the undamaged and damaged cases as an approximation. Their results indicated that this method was robust in locating single or multiple damage locations in structures. The disadvantage of this method is the need for the elemental stiffness matrix.

Stubbs *et al.* (1995) successfully applied a method based on the decrease in modal strain energy on a damaged bridge. They defined the strain energy in terms of mode shapes for a particular mode ϕ_i of a beam with length l, as:

$$\frac{1}{2}\int_0^l EI\left(\frac{\partial^2 \phi_i}{\partial x^2}\right)^2 . \tag{3.14}$$

For a linearly elastic beam structure, they introduced the strain energy damage index of N modes for the pth element, β_p, as:

$$\beta_p = \left(\sum_{i=1}^{N} \mu_{ip}^{d} \right) \Big/ \left(\sum_{i=1}^{N} \mu_{ip}^{u} \right), \tag{3.15}$$

where μ_{ip} terms are the values of the fractional strain energy for mode *i* between the endpoints of element *p*, denoted by *a*, *b*. They expressed these fractional strain energies for undamaged and damaged beam, denoted by *u* and *d*, respectively, as:

$$\mu_{ip}^{u} = \left(\int_a^b \left(\frac{\partial^2 \phi^u}{\partial x^2} \right)^2 dx + \int_0^l \left(\frac{\partial^2 \phi^u}{\partial x^2} \right)^2 dx \right) \Big/ \int_0^l \left(\frac{\partial^2 \phi^u}{\partial x^2} \right)^2 dx, \tag{3.16}$$

$$\mu_{ip}^{d} = \left(\int_a^b \left(\frac{\partial^2 \phi^d}{\partial x^2} \right)^2 dx + \int_0^l \left(\frac{\partial^2 \phi^d}{\partial x^2} \right)^2 dx \right) \Big/ \int_0^l \left(\frac{\partial^2 \phi^d}{\partial x^2} \right)^2 dx. \tag{3.17}$$

Higher values of β_p indicate members that are probably damaged. Assuming that the collection of the damage indices β_p represents a sample population of a normally distributed random variable, a normalized damage index is obtained using the following formula:

$$Z_p = \left(\beta_p - \overline{\beta}_p \right) / \sigma_p, \tag{3.18}$$

in which $\overline{\beta}_p$ and σ_p represent the mean and standard deviation of the damage indices, respectively. They assumed that the normalized damage indices with values greater than two are associated with potential damage locations. The advantage of this method is that the modes do not need to be mass normalized which makes it very useful when using ambient vibration. The details of damage localization algorithm are presented in Appendix-A.

Petro *et al.* (1997) showed that the strain energy index has a significant change in the vicinity of damage in a plate girder of a bridge. Cornwell *et al.* ((1997), (1999)) applied this damage index to plate structures. The algorithm was found to be effective in locating areas with stiffness reduction as low as 10% using relatively few modes. However, Cornwell *et al.* (1998) concluded that this method could identify two damage locations with the same size, but when the two damage locations had different levels of damage, the algorithm tended to only identify the location with the largest amount of damage. Wang *et al.* (1998) noticed the same observation in monitoring a suspension bridge. Guan and Karbhari (2008) presented a damage localization technique based on an energy criterion. It is shown that the proposed method exhibits superior performance under noisy conditions and sparse measurements compared with some traditional techniques based on modal curvature.

3.2.7 Modal Flexibility Method

Pandey and Biswas (1994) demonstrated the so-called flexibility matrix method using the mass normalized measured mode shapes and frequencies as follows:

$$[F] \approx [\phi][A]^{-1}[\phi]^T , \tag{3.19}$$

where $[F]$ is the flexibility matrix, $[\varphi]$ is the measured mode shape, and $[A] = diag(\omega^2)$ is the matrix of eigenvalues of the structure. The formulation of the flexibility matrix in Eq. (3.19) is approximate because not all the modes will be measured. However, because of the inverse relationship to the square of the modal frequencies, the measured flexibility matrix is most sensitive to changes in the lower-frequency modes of the structure. From the pre- and post-damage flexibility matrices, a measure of the flexibility change $[\Delta F]$ caused by the damage can be obtained from the difference of the respective matrices.

This method was applied to an actual beam, Pandey and Biswas (1995). Results of experimental examples showed that damage location could be obtained from the first two modes of the structure. However, there were false positive (the prediction of locations that were not damaged) near the ends of the beam, due to the free-free boundary condition. Using sensitivity analysis, Zhao and DeWolf (1999) showed that the modal flexibility is more sensitive to damage than either natural frequencies or the mode shapes.

Farrar and Jauregui ((1998-a), (1998-b), respectively) give experimental and numerical comparative studies of damage algorithms on a bridge. They stated that the flexibility matrix method derived from modal properties needs further studies to assess the effects of assuming an identity mass matrix when normalizing the mode shapes obtained from ambient tests.

3.2.8 Modal Stiffness Method

A variation on the use of the dynamically measured flexibility matrix is the use of the dynamically measured stiffness matrix, defined as pseudoinverse of the dynamically measured flexibility matrix. Similarly, the dynamically measured mass and damping matrices can be computed. Mannan and Richardson (1990) used direct comparison of these measured parameter matrices to estimate the location of damage. They found that the higher modes are most important to locate damage. Salawu and Williams (1993) found that using direct changes in the aforesaid matrices was the best among four different methods. The other three methods used the model-updating procedures.

Zimmerman and Kaouk (1994) have developed a damage detection method based on changes in stiffness matrix that is derived from measured modal data. The eigenvalue problem of the damaged structure is formulated as:

$$\left[\left[K - \mathit{\Delta} K\right] + \lambda_i'\left[M - \mathit{\Delta} M\right]\right]\left\{\varphi_i'\right\} = \left\{0\right\}, \tag{3.20}$$

where ΔK and ΔM represent the perturbations in the stiffness and mass matrices, respectively. Two expressions of a damage vector, $\{D_i\}$, for the ith mode can then be obtained by separating the original matrices from those containing the perturbation matrices and hence,

$$\left\{D_i\right\} = \left(\left[K\right] - \lambda_i'\left[M\right]\right)\left\{\varphi_i'\right\} = \left(\left[\mathit{\Delta} K\right] - \lambda_i'\left[\mathit{\Delta} M\right]\right)\left\{\varphi_i'\right\}. \tag{3.21}$$

To simplify the investigation, damage is considered to alter only the stiffness of the structure, i.e., $\Delta M = 0$. Therefore, the damage vector reduces to**:**

$$\left\{D_i\right\} = [\mathit{\Delta} K]\left\{\varphi_i'\right\}. \tag{3.22}$$

In a similar manner as the modal flexibility matrices previously defined, the stiffness matrix $[K]$ can be approximated from incomplete mass-normalized modal data as:

$$[K] \approx [\varphi][A][\varphi]^T. \tag{3.23}$$

The difference between undamaged and damaged matrices, $[\Delta K]$, can be easily obtained by subtraction. Then, the damage vector for the i th mode, $\{D_i\}$, can be obtained using Eq. (3.22). Again, the formulation of stiffness matrix in Eq. (3.23) is an approximation because not all the modes will be measured.

3.2.9 Environmental Effects on Dynamic Characteristics

In structural health monitoring using vibration-based damage detection methods, data acquisition could take a significant amount of time using conventional or advanced measurement devices for large structures. During that time, changes in structural characteristics can occur due to environmental changes (temperature, humidity, wind, etc.), which cause large variations in the vibration response at the same structural point. During damp weather, for example, concrete bridges are reported to absorb considerable amount of moisture, which increases their masses and alters their natural frequencies. Metwally *et al.* (2001), found that the natural frequencies are affected noticeably by the wind speed especially, the low frequency modes. Farrar *et al.* (1997) found that normal environmental changes could account for changes in frequencies of the Alamosa Canyon Bridge as much as 5% over a 24 hour time period, which may be larger than those induced by structural damage. Sohn *et al.* (1999) recommended further tests during different times of the year as well as different times of the day. Unfortunately, analytical investigations studying the variability of dynamic characteristics caused by changing environmental conditions have been lacking. Abdo (2003) has done an analytical investigation of the variations in natural frequency caused by typical changes in temperature. The effects of both the Young's modulus and the structural dimensions (via thermal expansion) on the structural frequency are thoroughly studied. The results indicate that the natural frequencies of beam and bar elements decrease with increasing temperature. Also, the Young's modulus is the dominant factor influencing the variations in natural frequency due to temperature changes.

Abdo (2005) investigated numerically the variations in structural modal frequencies caused by typical short or long-term temperature changes. The investigation covers the application of uniform and non-uniform temperatures in truss and beam structures. Also, the effect of changes of ambient temperatures on modal frequencies of structures with different boundary conditions is thoroughly studied. The results indicate that natural frequencies shifts of truss/beam structures are inversely proportional to uniform changes in temperature in the feasible range of a typical test environment with a maximum value of 1% during the year. Also, the frequency shifts are the same for truss/beam structures with different boundary conditions, in spite of the fact that stiffer supports increase the lower frequencies of these structures significantly. On the other hand, non-uniform change in temperature of a structural system has lower effect on its modal frequencies than that due to uniform change in temperature. Also, it has been demonstrated that changes in natural frequencies due to changes in surrounding temperature may mask the effect of small structural damage and consequently may lead to inaccurate results of damage detection techniques. Thus, it is important to take steps to stabilize the ambient temperature environment prior to and during taking of modal data measurements of truss/beam structures. Otherwise, the influence of ambient temperature should be taken into account.

Because little is known concerning its nature, the effect of dead loads on natural frequencies of structures is ignored in most previous beam studies. Abdo (2008) has investigated the effect of dead loads on the natural frequencies of beams. Based on the numerical results, it is shown that both distributed and concentrated loads have considerable influence on the lower frequencies of beams, especially the fundamental one and this effect increases with the increase of dead load. However, the effect of dead load may be neglected for frequencies higher than the first five natural frequencies. Also, the effect of dead load is large for single span beam and is smaller for multi-equal-span beams with all spans loaded with the same load value. Furthermore, in multi-equal-span beams, the case of loading which provides maximum positive bending moment between beam-supports leads to maximum change in frequencies. However, loading adjacent spans diminish the effect of dead load on beam frequencies and consequently, the case of loading which provides maximum negative bending moment at supports leads to minimum change in frequencies.

Guan and Karbhari (2008) found that the seasonal variation of temperature can have a significant impact on natural frequencies. At the same time, the effect of damage, if any, on the natural frequencies seems to be small and may not be observable compared with the greater effect of varying environmental conditions. They state that the study of environmental effects has gradually drawn more attention but much work still remains to be done.

Fan (2011) studied the temperature effect on dynamic response of fiber reinforced polymer (FRP) sandwich beams/panels for condition assessment. A series of FRP sandwich beams and sandwich panels were investigated for dynamic response change under temperature effect based on the material data obtained from dynamic mechanical analysis. He found that the temperature effect may only introduce a 2%-3% modal frequency shift to the FRP sandwich beam/panel over a 100 ℃ temperature change.

3.3 Static Response-Based Method

One of the global damage detection methods is using static response-based method. In this method, the static responses of the structural system are measured due to static loading. Since the static responses of a structural system are functions of the structural parameters, these parameters may be identified by using the changes in static characteristics. One of the consequences of the development of damage is the decrease in local stiffness, which in turn results in changes in some of the responses.

Significant work has been done in the formulation of vibration-based damage detection algorithms, e.g., natural frequencies, mode shapes, mode shape derivatives, and modal stiffness and flexibility matrices, as discussed in Section 3.2. Other research efforts have been devoted to damage detection using combined data of static and modal tests for damage identification (Lee *et al.*

(2010), Oh and Jung (1998), and Oh *et al.* (1999)). Since the static equilibrium equation is solely related to the structural stiffness, and accurate static displacement and strain data can be obtained rapidly and cheaply, the static damage identification methods have recently attracted relatively more attention. Banan *et al.* ((1994-a), (1994-b)) proposed an algorithm for estimating member constitutive properties of the finite element model from measured displacements under a known static loading. The algorithm was based on the concept of minimizing an index of discrepancy between the model and the measurements using the constrained least-square minimization. Hjelmstad and Shin (1997) proposed an adaptive parameter grouping scheme to localize damage in a structural system for which the measured data are sparse. The algorithm can evaluate the sensitivity of each member parameter simultaneously with the process of damage detection, but requires much computation due to the number of perturbation trials used in the algorithm. Sanayei and Saletnik (1996) presented a method for structural parameter identification utilizing elemental strain measurements. They state that strain measurements are more accurate than ordinary displacement measurements and can easily be used on bridges, buildings, and space structures. They succeeded to get accurately the element stiffnesses of two-dimensional truss and frame structures, which could be used for damage assessment of structures. Chen *et al.* (2005) proposed a two-stage damage detection approach based on using grey system theory to localize damage by only using the structural measured static displacement on a cantilever beam. They showed that the grey-relation-analysis-based method can localize slight to moderate damages and can identify the damage magnitude with satisfactory accuracy. Unfortunately, they did not mention the sensitivity of damage to number of load cases, intensity of loading, limited measured static data, and the application of this approach on large-scale and complex structures.

Indeed, among different approaches developed for structural damage detection, changes in displacement curvatures have shown promise for locating structural damage, Chen *et al.* (2005). An analytical study of the relationship between damage characteristics (location and severity) and changes in displacement curvature was presented by Abdo (2012). A parametric study was carried out on using changes in displacement curvature in structural damage detection. Thus, the influence of many parameters such as: number of measurement data, cases of loadings, intensity of loading, and measurement noise, were investigated. To demonstrate the results, numerical analyses of two examples were studied: an overhanging beam (statically determinate structure) and a two-span continuous beam (statically indeterminate structure) with different damage characteristics. A careful numerical study was carried out using the finite element method to analyze static behavior of the two-span continuous beam. The merits as well as the disadvantages of using displacement curvature in structural damage detection were discussed. Abdo (2012) has drawn the following conclusions:

1. Changes in displacement curvatures (derived from measured static response only) has the characteristic of localization at the damaged region and can be used as a good damage indicator even for a small amount of damage; using only one case of loading. Indeed, this method is promising since it is cheaper and more accurate than vibration-based damage detection methods which needs expensive sensors and has a lot of noise if ambient vibration is used.
2. The number of measurements is not important, but the most important thing is the data or measurement at or near damage locations and accuracy of measurements.
3. It is possible to use one case of loading to detect and localize different damage characteristics using GRC; on condition that this case of loading produces bending moment all over the beam length. Fortunately, most of

damages occur at locations which have bending stresses.

4. Using GRC as a damage localization approach does not depend on the intensity of loading but depends on the case of loading.
5. Measurement noise has negligible effect on the damage localization approach and GRC is a good damage localization technique. However, GRC does not give an indication about the magnitude of damage.

Indeed, the above results prove that the changes in displacement curvatures are promising in detecting and locating structural damage. Abdo (2012) expects that if displacement curvatures are measured directly without approximation (Kovacevic *et al.* (2008)), they will give more accurate results of damage identification. The basics of Grey relation coefficient (GRC) and some results of using displacement curvatures are found in Appendix-B.

3.4 Physical Model-Based Method

When a finite element model of the structure is utilized in the damage identification process, such physical model-based methods are sometimes also referred to as finite element model updating (FEMU) based methods. Indeed, the process of finding a model from data is called system identification. General system identification is a research branch of electrical engineering. An authoritative reference regarding system identification is the book of Ljung (1999).

Caesar (1986) has given a very comprehensive review on the optimal matrix update approach. An excellent textbook discussing the finite element model updating techniques and their applications is due to Friswell and Mottershead (1995). Lim and Kashangaki (1994) compared the best achievable eigenvectors with the measured modes to detect the damage in a space truss. Ruotolo and

Surace (1997) fit a mathematical model to the modal parameters of the lower modes for damage detection and sizing of cracks in a steel beam. Also, Kosmatka and Ricles (1999) could detect the damages inflicted to a space truss using analytical model that is correlated to the experimental baseline data. Abozeid *et al.* (2006) used the experimental modal testing of the Suez-Canal cable-stayed bridge to obtain the dynamic characteristics of the bridge. He could extract mode shapes and corresponding natural frequencies which were very near to those extracted theoretically using finite element analysis. Thus, regular updating can be used to assess the status of the bridge.

3.4.1 Difficulties of FEMU Approach

Previous researchers have shown that FEMU can be an extremely useful technique for damage identification under certain conditions. But it should also be noted that some difficulties still exist when implementing FEMU technique in a vibration-based structural health monitoring system. It is not always an easy task to guarantee the accuracy and validity of a finite element model. Some of the current challenges involve the non-uniqueness of the solution, ill-conditioning of the identification problem, and numerical convergence problems of the optimization algorithm. Firstly, due to the fact that only a small number of degree-of-freedoms can be measured experimentally and there exist a large number of uncertain parameters to be updated, the updating problem is usually ill-conditioned. This directly leads to the second problem of numerical convergence difficulties. A small amount of noise in the measured structural response can sometime corrupt the result to a great extent. That is the reason that some researchers state that this approach is expensive and time consuming, Abdel-Wahab and De Roeck (1999).

3.4.2 Suggestions to Improve FEMU Approach

Although some of the above challenges are inherent to the inverse identification problem to which the FEMU problem belongs, others can be solved or alleviated through the use of appropriate techniques. Indeed, the choice of proper structural parameters to update is one of the main difficulties of FEMU. Careless choice of parameters usually leads to ill-conditioned identification problem. It is suggested that the ill-conditioning of the FEMU problem can be greatly alleviated by using the information about the damage location. Such information can be obtained from damage localization procedures such as the modal curvature techniques or element strain energy damage indices. Also, dense sensor networks could be used to improve the spatial dimension of the measured data, thus reducing the ill-conditioning of the problem. Dynamic properties other than mode shapes and frequencies could also be used to provide higher sensitivity to structural changes. Globally, robust optimization algorithms can be adopted to alleviate the convergence problem, Guan and Karbhari (2008). Again, model-based damage detection methods are capable of estimating both the location and absolute severity of the damage and provide at least level III of structural damage identification.

3.5 Implementation of Long-Term SHM

Xia *et al.* (2006) investigated the variation of frequencies, mode shapes and damping with respect to temperature and humidity changes. A reinforced concrete slab, which was constructed and placed outside the laboratory, has been periodically vibration tested for nearly two years for the first four modes. They found that the frequencies have a strong negative correlation with temperature and humidity, damping ratios have a positive correlation, but no clear correlation of mode shapes with temperature and humidity change can be observed. Linear regression models between modal properties and environmental factors are built. A quantification analysis shows that variation of the elastic modulus of the material is the primary cause of the variation of modal properties.

A long-term structural health monitoring (SHM) system was installed on the Kings Stormwater channel composite bridge (California, USA) shortly after it was opened to traffic in May 2001. The bridge is a two-span highway bridge with a total length of approximately 20.1 m and a width of approximately 13.0 m, as shown in photo 3.1. The superstructure is of slab-on-girder type, with two equal spans of 10.0 m, and a cap beam connecting two adjoining spans. Abutments on both ends separate the bridge from the road approach. The SHM system was designed and implemented according to the vibration-based SHM. The primary purpose of the system is to monitor the changes in performance of the structure, caused by vehicular loading and environmental effects, and to provide early warning of conditions that might affect structural integrity. The system also serves the purpose of rapid condition evaluation after major events such as earthquakes and floods, Guan and Karbhari (2008). They used operational modal analysis to extract modal parameters from the collected ambient vibration responses of the structure. The identified modal parameters are used as input for damage localization algorithm. The possible damage regions obtained through damage localization helps to narrow down the number of unknown parameters used in the finite element model updating process. A baseline finite element model well-correlated with experimental modal data is obtained through FEMU. The results of the subsequent updating provide detailed information regarding the location and severity of the damage. Their results appear to agree with the visual inspection results. It is worth to mention that they used combined response-based and model-based methods for structural damage detection.

Wireless sensor networks (WSNs) have the potential to offer significant advantages over traditional wired monitoring systems in terms of sensor, cabling, and installation costs as well as expandability. To evaluate the potential of WSNs for use in bridge management, Hoult *et al.* (2010) installed a network of seven sensor nodes on a three-span reinforced concrete slab bridge, in the UK known as the Ferriby Road Bridge, as shown in Photo 3.2. The seven sensor nodes were as follows: three displacement transducer nodes were placed on the soffit of the bridge to measure the change in crack width, three inclinometer sensor nodes were mounted on two of the elastomeric bearing pads to measure the change in inclination of the bearing pads, while a final node monitored temperature. They found that WSNs offer a more cost-effective alternative to conventional wired monitoring systems making them an appealing option for both short and long-term monitoring. For short-term monitoring, the total cost per node for WSNs is US$50 versus US$3,000 for the wired system. However, for long-term monitoring the total cost per node for WSNs is US$580 versus US$3,000 for the wired system. They state that there are savings to be gained for any monitoring application if WSNs are used and they should be considered as possible replacement for wired monitoring systems. However, there are issues in terms of radio connectivity as well as battery life and battery size that must be taken into consideration before choosing a particular WSN solution.

Photo 3.1 *Kings Stormwater channel composite bridge (California, USA), Guan and Karbhari (2008).*

Photo 3.2 *Ferriby Road Bridge in the UK, Hoult et al. (2010).*

3.5.1 Data Processing and Archiving

The automated collection and digitization of data are often controlled by two collaborating programs. The first program serves the collection and digitization function. The second program serves both data transmission and analysis functions. Once collected, event-based data are analyzed to extract vibrational signatures of the structure. All raw event-based data and the extracted signatures are then archived in a database for future reference, Guan and Karbhari (2008).

It should be mentioned that the amount of data collected is directly proportional to the sample rate and the number of sensors or channels simultaneously measured. Thus, in order to achieve continuous monitoring of a structure, tens of thousands of samples may be recorded every second and the data acquisition, transmission and archiving system must be designed to accommodate that.

3.5.2 Future of Long-Term SHM

Based on the review of the existing literature in Chapters 2 and 3, it can be concluded that long-term vibration-based health monitoring of civil structures is still in its infancy. This can be clearly seen from the rarity of successful real world applications.

Indeed, current health monitoring systems e.g., (Abe *et al.* (2000), Lee *et al.*

(2010), Miyashita and Nagai (2008), and Hoult *et al.* (2010)) place more emphasis on monitoring of local structural behavior such as strain, stress, force and temperature rather than the global dynamic response of the structure. Although local structural behavior can be a useful indicator of the health condition of the structure, such monitoring system provides no information about the global behavior of the structure and will face difficulty in accomplishing tasks of estimating the remaining capacity and usable life. Furthermore, among the large number of papers in the literature on the topic of SHM, only a small number of papers deal with the severity and the remaining capacity of the structure (levels III and IV of damage identification). The majority are limited to the modest goal of discovering the occurrence and the location of the damage. Thus, new approaches with improved performance under real world situations are needed.

Lastly, on a broader scale, the current approaches to the SHM problem can be divided into two distinct areas: (1) using structure-dynamic properties to detect structural changes at global level, and (2) using local NDE methods to locate and quantify damages in local components. Both approaches have their own advantages and limitations. Neither alone can satisfy all the stringent requirements from the end users. A new multi-level structural health monitoring system integrating global and local-level diagnostics needs to be developed. Global-level techniques can be used to provide rapid condition screening, locate the proximities of the anomalies and evaluate their influence on global structural behavior, while local NDE techniques can be applied to the identified damage region in order to better define the location and severity of the damage and its effect on local components. Mission-tailored new sensor technologies such as FOSs with wireless communication capabilities will be essential to reduce the system cost and improve structural health monitoring efficiency, Guan and Karbhari (2008).

Chapter 4

Response Measurements and Their Applications

4.1 Introduction

The experimental setup used for vibration testing is basically quite simple although there exists great many different variants on it, in terms of the specific items used. There are three major items, Ewins (2000) as follows:

1. An excitation mechanism;
2. A transduction system (to measure the various parameters of interest);
3. An analyzer, to extract the desired information (in the presence of the inevitable imperfections which will accumulate on the measured signals).

The structure can be excited into vibration in several ways; forced vibration (shaker, hammer blow, etc.) or ambient vibration (wave, wind, roadway excitation, and micro-earthquakes). In fact, forced-vibration testing is preferred to ambient testing because it produces data from which system parameters can be better identified, Reynolds and Pavic (2001). However, ambient vibration measurement is one of the most convenient vibration measurement methods for large civil structures, since it can be done under service condition and no specific device is required for excitation, Abe *et al.* (2000).

There are great many different possibilities for the devices available to measure the excitation forces and/or the various responses of interest. Different types and advances of the devices used for measurement of response will be discussed in next sections.

The function of the analyzer is simply to measure the various signals developed by the transducers in order to ascertain the magnitudes of the excitation force(s) and/or responses. There are different types of analyzer available and the choice will depend on the type of excitation that has been used.

4.2 Strain Gauges

Strain gauges are the first devices used for measuring static and dynamic responses. The use of strain gauges is based on the fact that the resistance of a conductor changes when the material of the conductor is subjected to strain. The practical application of the above principle consists simply in bonding a resistive wire on an insulating backing to the object the strain of which is to be determined. However, the strain can be generally read off directly from the meter that measures the resistance variations of the strain gauges, Potma (1967).

If we consider *R* as the resistance of the filament, (which is generally made of 'Constantan' an alloy of a high resistivity), and ΔR is the increase in the resistance. There is an almost linear relation between the strain ε and the resistance variation. The proportionality constant is called the *K* factor:

$$\frac{\Delta R}{R} = K\frac{\Delta l}{l} = K\varepsilon \,. \tag{4.1}$$

The value of *K* is generally 2-4 and depends on the material used for the filament. Semiconductor strain gauges, which have silicon strip as resistive material, have higher *K* factors (up to about 180). There are many types of strain gauges such as wire gauges, rosette gauges (for measuring the magnitudes and directions of principal stresses), etc. Also, there are special purpose strain gauges such as, stress gauges, flexagauges (for measuring bending stresses), etc. The details about these strain gauges can be found in Potma (1967).

4.3 Gyroscopes

Gyroscopic sensors are commonly used in control systems for stabilizing vehicle systems, De Silva (1989). They have been successfully used in airplanes, space stations, ships, and land vehicles. If we consider a rigid body spinning about an axis at angular speed ω, and if the moment of inertia of the body about that axis is J, the angular momentum H about the same axis is given by:

$$H = J\ \omega. \tag{4.2}$$

Newton's second law (torque=rate of change of angular momentum) tells us that to rotate (precess) the spinning axis slightly, a torque has to be applied, because precession causes a change in the spinning angular momentum vector (the magnitude remains constant but the direction changes). This is the principle of operation of a gyroscope. Gyroscopes can measure angular displacements and rate gyro (a gyroscope provided with a torsional spring and a viscous damping) can be used to measure angular speeds, De Silva (1989).

It is worth to mention that the fiber optic gyroscope is an angular speed sensor that uses fiber optics. Contrary to the implication of its name, however, it is not a gyroscope. Two loops of optical fibers wrapped around a cylinder are used in this sensor. One loop carries a beam from monochromatic light (or laser) beam in the clockwise direction; and the other loop carries a beam from the same light source in the counterclockwise direction. The difference in frequencies of the two laser beams received at a common location will measure the angular speed of the cylinder. More details of this type of sensor can be found in De Silva (1989).

4.4 Piezoelectric Transducers

Although strain gauges are often found to be convenient because of their minimal interference with the test object, the piezoelectric type of transducer is by far the most popular and widely-used means of measuring the characteristics of interest in vibration testing. The basic principle of operation makes use of the fact that an element of piezoelectric material (either a natural or synthetic crystal) generates an electrical charge across its end faces when subjected to a mechanical stress. By suitable design, such a crystal may be incorporated into a device, which induces in it a stress proportional to the physical quantity to be measured (i.e., force or acceleration), Ewins (2000).

One of the advantages of piezoelectric transducer is that it is an active device, and does not require power supply in order to function. However, there is a low frequency limit below which measurements are not practical. This limit is usually determined not simply by the properties of transducer itself, but also by those of the amplifiers, which are necessary to boost the very small electrical charge that is generated by the crystals into a signal strong enough to be measured by the analyzer, Ewins (2000).

Conventional transducers such as piezoelectric accelerometers must be attached directly to the structure, which cause lack of measurement points. Furthermore, this attachment introduces localized mass and adds local damping which distorts the dynamic response of the structure in the vicinity of damage. These effects may be significant, especially if the transducer is relatively large or heavy (relative to the effective mass of the structure at the point of attachment), Woon and Mitchel (1996). So, this has raised the need for non-contacting devices that can provide a more accurate depiction of the dynamic response.

4.5 Laser Measurements and Their Applications

The introduction of LASER, (Light Amplification by Stimulated Emission of

Radiation), in sensing and gauging applications has revolutionized dynamic testing and analysis in recent years. A laser can provide single frequency (monochromatic) light source. LASER can be used directly as in Laser Doppler Vibrometer (LDV), or indirectly as in the fibre optic sensors.

4.5.1 Laser Doppler Vibrometer (LDV)

The basic LDV transducer is an optical measurement device, which is capable of detecting the instantaneous velocity of the surface of the structure. The velocity measurement is made by directing a beam of laser light at the target point and measuring the Doppler-shifted wavelength of the reflected light, which is returned from the moving surface, using an interferometer. The main requirement is for a line of sight to the target measurement point and a surface that is capable of reflecting the laser beam adequately. Characteristics for typical LDV device can be found in Ewins (2000).

The major advantage of LDV is the non-intrusive nature of their operation. Also, it possesses extremely high resolution and able to measure in wide frequency range. Subsequently, the technique has attained popularity in special situations requiring the use of a non-contacting optical sensor such as biomedical vibration analysis, Sriram *et al.* (1992). Furthermore, it can be applied for difficult response measurement tasks, such as taking measurements on curved surfaces and in hostile measurements, especially on surfaces which have such high temperatures that the conventional transducers cannot be employed. On the other hand, the main limitations of the LDV as a general-purpose response transducer are the line of sight requirement and the problems associated with speckle noise, a phenomenon which results in occasional drop-outs, or null measurements (the laser receives insufficient light to detect the instantaneous velocity).

Actually, LDV has attracted many researchers in civil engineering in both laboratories (Pardoen and Zang (1998) and Kaito *et al.* (2000)), and in the field (Kaito *et al.* (2001), Miyashita and Nagai (2008)). Kaito *et al.* (2000) have succeeded to decrease the speckle noise. By changing the irradiation angle for each repetitive measurement, per 0.01 degree around the first measurement point, the concentrated contents of the speckle noise are prevented. Next, the measured vibration results of data that contain more than five speckles are eliminated when carrying out averaging of the spectrum. Miyashita and Nagai (2008) developed and applied LDV to tensile force measurement of cables in a cable-stayed bridge. They showed that LDV has the ability to identify lower to higher natural frequencies. Also, they developed the system combining the LDV with the Total Station for automated remote measurement of the whole large bridge.

A natural extension of LDV is to incorporate a dynamic feature in the location mechanism. So, in the Scanning Laser Doppler Vibrometer (SLDV), it is possible to control the mirror drives so that the measurement point moves across the surface of the structure in a controlled and prescribed continuous manner. Indeed, the introduction of SLDV has revolutionized dynamic testing and analysis in recent years, Woon and Mitchel (1996). It is possible for SLDV to measure the vibration velocity at more than 60,000 locations on the scanned surface of a structure in less than one minute, Arruda (1993). SLDV is used in structural damage detection by many investigators, (Chen *et al.* (1998) and Khan *et al.* (1998)).

Recently, due to advantage of the high spatial density of SLDV, it has been applied to the rotational DOF measurement problem with promising results, Ewins (2000). Arruda (1993) has shown that for low-damped structures with well-separated modes, it is feasible to extract the angular degrees of freedom. Using a steel plate and measuring only the out-of-plane displacements, he has proposed that the measured forced operating shapes and the identified mode shapes can be curve fitted using 2-dimensional Fourier series, from which two angular degrees of freedom (DOFs) can be extracted, thus providing three DOFs at each measured location. Ratcliffe and Lieven (2000) have presented a technique to calculate the two out-of-plane rotations, from experimental translation data. Mitchell *et al.* (1998) has given a description of the advances toward the implementation of a three dimensional, six-DOFs structural dynamics measurement system. The availability of this basic experimental measurement tool opens the door to many advances in experimental dynamics measurements. Thus, Sharma (2008) developed a damage detection technique for a damaged plate using SLDV. He used the curvatures of modes for estimation of plate strain energy and to formulate the Damage Measure, which is used as effective indicator of the damage location and its extent.

4.5.2 Fiber Optic Sensors

Fiber optic sensors (FOSs) have been the subject of considerable research for the past three decades, with initial applications focused on military and aerospace uses, Grattan and Meggitt (1999). The characteristic component in a fiber optic sensor is a bundle of glass fibers (typically a few hundred) that can carry light. Each optical fiber may have a diameter on the order of 0.01 [mm]. In the direct type of optical fiber sensor, if the condition of the sensed medium changes, the light-propagation properties of the optical fiber change, providing a measurement of the change in conditions, De Silva (1989).

In general, FOS with light weight and fine wires which are capable to use in long distances and immune to general environmental effects are suitable for health monitoring. Different types of FOSs along with the principle of operation are given by Udd (1991). However, some types of FOSs such as Fabry-Perot

Interferometric sensors (Extrinsic or intrinsic), Fiber Bragg grating sensors, and Long-Period grating sensors are commonly used in system identification and health monitoring. Claus and Lenahan (1998) discussed the principle of operation and fabrication process of each of the above four different types of FOSs. Generally, the advantages of fiber optics include insensitivity to electrical and magnetic noise (due to optical coupling) and safe operation in explosive and hazardous environments. The disadvantages include direct sensitivity to variations in the intensity of the light source and dependence on ambient conditions (dust, moisture, smoke, etc.).

One of the unique features of FOSs is the distributed sensing capability, which means that multiple points can be sensed simultaneously by a single fiber. Yu and Yin (2002) provided a brief summary on distributed fiber optic sensors. By taking advantage of this multiplexing capability, not only can the magnitude of a physical parameter be monitored, but also its variation along the length of the fiber can be measured. Thus, distributed sensing can be achieved, which not only makes FOSs more cost-effective but also very compact. Thus, many important applications such as structure fatigue monitoring (e.g., monitoring the performances of dams and bridges) can be implemented in an effective way.

(1) Laboratory Studies

A number of groups have performed laboratory studies that demonstrate the ability of FOSs to detect strain in civil structures. Escober *et al.* (1992) used embedded and bonded fiber-optic interferometric sensors (FOIS) for five years in concrete specimens. The results confirm the good characteristics of the FOIS in terms of sensitivity, reliability, and aging in concrete applications. Masri *et al.* (1994) got the same results when they attached FOSs to specific rebar elements within a three-dimensional reinforcement cage in a reinforced concrete beam-column assemblage. They concluded that properly installed FOSs not only can survive in the harsh environment involved in the embedment process, but also can yield accurate quantitative strain information. Furthermore, several studies have compared fiber optic strains with traditional foil or resistance strain gauges; among them are (Gusmeroli *et al.* (1994), Habel and Hillemeier (1995), Kaide *et al.* (1996), and Kim and Paik (1997)). When the two sensors were co-located, the results were in good agreements.

(2) Applications to Civil Structures

When compared with traditional electrical strain gauges used for strain monitoring of large composite or concrete structures, FOSs have several distinguishing advantages, Yu and Yin (2002), including:

1. A much better invulnerability to electromagnetic interference, including storms, and the potential capability of surviving in harsh environments.
2. A much less intrusive size (typically 125 μm in diameter - the ideal size for embedding into composites without introducing any significant perturbation to the characteristics of the structure).
3. Greater resistance to corrosion when used in open structures, such as bridges and dams.
4. A greater capacity of multiplexing a large number of sensors for strain mapping along a single fiber link, unlike strain gauges, which need a huge amount of wiring.
5. A higher temperature capacity with a widely selectable range.
6. A longer lifetime, which could probably be used throughout the working lifetime of the structure (e.g., >25 years) as optical fibers are reliable for long-term operation over periods greater than 25 years without degradation

in performance.

These features have made FOSs very attractive and suitable for quality control during construction and health monitoring after building large structures. Thus, in addition to the laboratory studies described above, FOSs have been used to measure strain and vibration in a number of newly constructed structures. They have also been attached to existing structures for non-destructive evaluation of structural health, such as crack growth, and monitoring use, e.g., traffic load on roadways. A state of the art of application of FOSs in concrete structures as well as in advanced fiber reinforced composites is given by Grattan and Meggitt (1999).

Yu and Yin (2002) demonstrated applications of FOSs to large civil structures; bridges and dams. For bridges, they showed the advantages and disadvantages of different FOSs for measuring temperature, static strain, and transient strain of both steel and concrete bridges. They stated that the choice of sensors is vital for the accuracy of different response measurements. Since dams are probably the biggest structures in civil engineering, hence it is vital to monitor their mechanical properties during and/or after construction in order to ensure the construction quality, longevity, and safety of the dam. They used a Brillouin-based distributed sensor to monitor temperature distribution in a concrete slab with dimensions of: 15m length, 10m width, and 3m height. These concrete slabs are used for raising the height of the dam in order to increase the power capability of the associated hydroelectric plant. They showed that the embedded fiber cable which is installed during the concrete pouring could give two-dimensional temperature distribution of the whole slab area at different times after concreting. They also showed that FOSs can be used in monitoring load and displacement changes in underground excavations of mines and tunnels.

Inaudi and Glisic (2006) used distributed FOSs for the monitoring of civil and industrial structures. The strain and temperature of two dams as well as an old gas pipeline are successfully monitored. They used sensors based on Raman and Brillouin scattering which able to measure strain or temperature variations of fibers with length up to 50 km with spatial resolution down in the meter range. Using a single optical fiber with a length of tens of kilometers they could obtain dense information on the structure's strain and temperature distribution. They showed that using an appropriate sensor design, it is possible to successfully install distributed sensors on large or elongated structures; such as dams, large bridges and pipelines and obtain useful data for the evaluation and management of the monitored structures.

Krebber *et al.* (2012) have developed technical textiles with embedded distributed FOSs for the purposes of structural health monitoring in geotechnical and civil engineering. The distributed FOSs are based on Brillouin scattering. Such "smart" technical textiles are used for reinforcement of geotechnical and masonry structures. They showed that embedded FOSs provide online information about the condition of the structure and about the occurrence and location of any damage or degradation.

(3) Curvature Gauges

Xu *et al.* (1994) developed a new bending gauge, using a pair of surface mounted fiber Bragg gratings with excellent agreement with the expected strain sensitivity. Djordjevich and Boskovic (1996) proposed a new fiber-optic gauge, which is sensitive to the deformation curvature of structures. Using such sensors, they could measure deformation curvatures with a diameter approaching 5 km. The advantage of these sensors is that curvatures can be measured anywhere, including along the neutral axis of the cross section where there is no strain in bending. Djordjevich (1998) extended the application of curvature gauge to include torsional and axial loading situations. Djordjevich *et al.* (2000) applied the curvature gauge to measure the mode shapes. The number of gauges required equals the number of dominant vibration modes of interest. The analytical mechanism of using curvature gauges to measure curvature directly is given by Kovacevic *et al.*, (2008). Indeed, application of the advances in FOSs in the field of damage detection and system identification will make it more reliable and applicable in near future.

(4) Future of Fiber Optic Sensors

Mission-tailored new sensor technologies such as piezoelectric and fiber optic sensors with wireless communication capabilities will be essential to reduce the system cost and improve efficiency. Thus, advances in sensors, computational and telecommunications technology, could provide for continuous and autonomous assessment of structural response, Guan and Karbhari (2008).

It is expected that in near future FOSs will be more cost-effective and consequently be implemented in structural health monitoring in an effective way. Thus, FOSs are promising in the field of SHM.

Chapter 5

Closure

Open Science

5.1 Conclusions

This book presents the state of the art of structural health monitoring (SHM): history, applications, and future. Based on the review of the existing literature in Chapters 2, 3 and 4, it can be concluded that long-term SHM of civil structures is still in its infancy. This can be clearly seen from the rarity of successful real world applications. The following conclusions are drawn:

1. Changes in frequencies, mode shapes, MAC, and COMAC imply the presence of damage but not the damage location. Also, these changes do not give an indication for a very small amount of damage, Abdo (2002).
2. Mode shape derivatives and modal strain energy have the characteristic of localization at the damage location(s). However, they are sensitive to noise because they are estimated using central difference approximation. They will be promising in SHM if these derivatives are measured directly without approximation, (Abdo (2002) and Abdo (2012)).
3. Changes in natural frequencies due to changes in environmental conditions (temperature, humidity, wind, additional dead loads, etc.) may mask the effect of small structural damage and consequently may lead to inaccurate results of damage identification. Thus, the environmental conditions should be taken into consideration during updating the health of the structure, (Abdo and Farghal (2005), Farrar *et al.* (1997), and Sohn *et al.* (1999)).
4. Ill-conditioning of the FEMU problem can be greatly alleviated by using the information about the damage location. Such information can be obtained from damage localization procedures such as the modal curvature techniques or element strain energy damage indices. Also, dense sensor networks could be used to improve the spatial dimension of the measured

data, Guan, and Karbhari (2008).

5. Model-based methods are generally capable of estimating existence, location, and absolute severity of the damage. However, response-based methods are typically limited to the existence and location of the damage when applied to civil engineering structure, Guan, and Karbhari (2008).
6. In continuous monitoring of a structure, huge amount of data may be recorded every second and the data acquisition, transmission and archiving system must be designed to accommodate that.

5.2 Future Perspectives

Global damage detection methods such as response-based and model-based methods have shown promise and they can be effective and reliable indices to identify structural damage(s). The following areas are suggested by the author for possible future researches related to SHM:

1. Robust optimization algorithms are needed to alleviate the convergence problem of FEMU and the impact of noise on static and/or dynamic response measurements.
2. New advanced sensor technologies such as piezoelectric and fiber optic sensors with wireless communication capabilities will be essential to reduce the system cost and improve SHM efficiency.
3. Efforts are needed to separate the changes in responses of structures due to damage from those due to environmental conditions.
4. For successful SHM, a new multi-level system integrating global and local-level diagnostics needs to be developed. Global-level techniques can be used to provide rapid condition screening, while local NDE techniques

can be applied to the identified damage region in order to better define the location and severity of the damage.

5. Implementation of advanced sensors to SHM of critical civil structures all over the world. Some of the critical structures in Egypt are: The Aswan High Dam and The Suez-Canal (Al-Salam) bridge.

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Appendix-A Damage Localization using Vibration-Based Method

A.1 Damage Localization Algorithm

We have known that the curvatures of mode shapes are sensitive and localized at the damaged region in Section 2.4.1. Now, we need to make a decision whether a structure is undamaged or damaged at a given location. In this study, the algorithm which is developed by Gibson and Melsa (1975), is used to calculate the damage indices that are functions of measurable pre- and post-damage modal characteristics. The damage localization is accomplished in four steps as follows, Stubbs et al. (1995):

1. Compute the curvature of mode shapes using Eq. (3.9).
2. Compute the absolute differences of the curvature at the i th node of the j th mode, β_{ij}.
3. Normalize the values of the indices according to the following rule:

$$I_{ij} = \left(\beta_{ij} - \mu_{\beta j}\right) / \sigma_{\beta j} \ , \tag{A.1}$$

   where, $\mu_{\beta j}$ and $\sigma_{\beta j}$ are respectively, the mean and standard deviation of the damage indices for the *j* th mode.

4. Classify the damage location at any point, *i*. The damaged location is that at which the damage indices have values $I_{ij} \geq 2$.

It should be mentioned that the points that have damage indices, $I_{ij} \geq 2$ indicate that the probability of false alarm (Pfa) is just 0.0228.

A.2 Some Results of Damage Localization

(1) Numerical Analysis

As a large statically indeterminate structure, the structure chosen in this study is a two-span continuous steel beam. The beam is assumed to have prismatic cross section and supported on a hinged support in the middle and roller supports at both ends, (Abdo (2004), (2012)). The pre- and post-damage modal parameters are calculated numerically using finite element method, where two-node beam element with six degrees of freedom per node is used. The finite element model of the beam consists of 60 equal-length 2-D beam elements and 61 nodes as shown in Fig. A.1. The cross-sectional area of the beam and the moments of inertia are, A=0.07 [m^2], and I_z= 0.040 [m^4], I_y= 0.001 [m^4], respectively, and the mechanical properties are, modulus of elasticity, E=210 [GN/m^2], Poisson's ratio, ν=0.3, and the density, ρ=7,850 [kg/m^3].

The eigenmodes calculated are orthogonal with respect to the inertia matrix M, i.e., $\varphi_i^T M \varphi_i = 1$. Only the translational displacement mode in the y-direction (u_y) is considered in the analysis. This was done because, in any experimental work, in general, rotations are not measured because of the difficulty in their measurements. Moreover, since we are interested only in the flexural modes of vibration, translation along the x-axis can also be neglected. For the sake of comparison and to be applicable to experimental data, the displacement modes are normalized with respect to the square root of sum of squares, (SRSS).

The damage of the model in this study is assumed to affect only the stiffness matrix but not the inertia matrix in the eigenproblem formulation. This assumption is consistent with those used by Banks *et al.* (1996), Pandey *et al.* (1991), and Abdo (2002). The change in the stiffness due to damage is modeled by a reduction in the modulus of elasticity of the element. The degree of damage is then related to the extent of reduction of the modulus of elasticity, *E*, of some elements. A linear modal analysis is performed to examine the robustness of curvature techniques in damage detection.

Six cases of damages are studied. These cases of damage are investigated to represent not only different locations but also different severities of damage. For each case of damage, the first five natural frequencies and the corresponding mode shapes are calculated. Table A.1 shows the damage characteristics of the six cases of damage. The first three cases of damage represent single damage with 30% reduction in modulus of elasticity, *E*, with different locations in elements No. 1, 8 and 15, respectively. The fourth and fifth cases of damage represent single damage with different severity of damage, 10% and 30% reduction in *E* of element No. 30. The sixth case of damage represents multiple damage locations with 30% reduction in *E* in elements No. 1, 8, 15, and 30, simultaneously. The four elements No. 1, 8, 15, and 30 are chosen to simulate different types of damage scenarios. Damage in element No. 1 represents a failure due to shear at the end support where the displacement and curvature have minimum values. Damage in element No. 8 represents a failure due to combined shear and bending moment where the displacement and curvature have small values. Damage in element No. 15 represents a failure due to bending moment where the displacement and curvature have maximum values. Damage in element No. 30 represents a failure due to combined shear and bending moment where the displacement has minimum value and the curvature has maximum value. The first five cases of damage represent single damage location with different degrees of severity of damages, whereas *Case*-*6* is studied to show the capability of the damage detection methods to detect and pinpoint multiple damage locations. More details of the model are found in Abdo (2004).

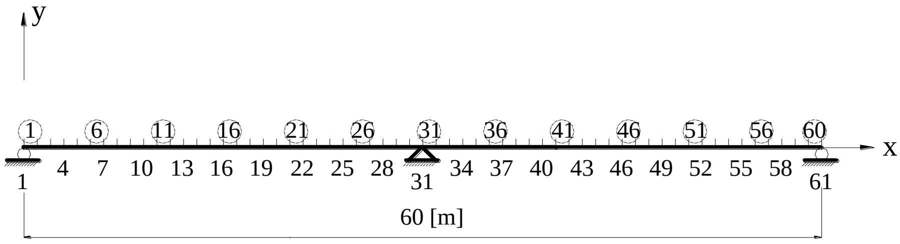


Figure A.1 *Geometry of the two-span continuous beam.*

Table A.1 *Damage characteristics of the beam model.*

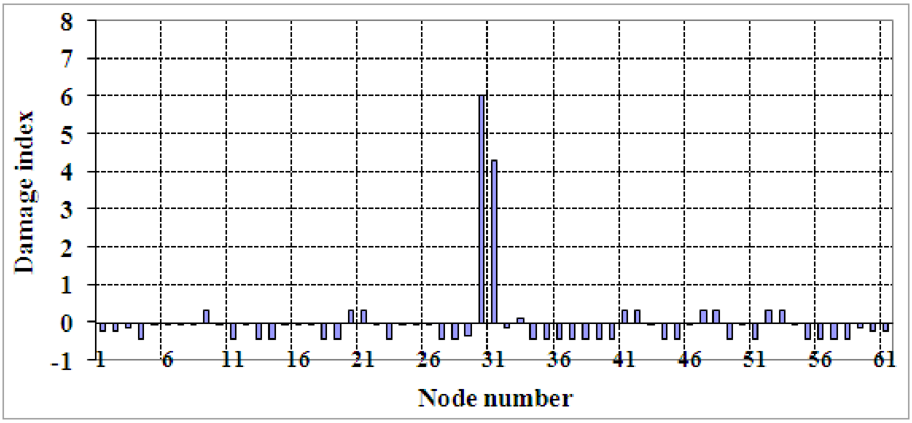
Case of damage	Damage location (Element No.)	Percentage reduction in E	Number of damage locations
Case-1	1	30	1
Case-2	8	30	1
Case-3	15	30	1
Case-4	30	10	1
Case-5	30	30	1
Case-6	1,8,15,30	30	4

(2) Results of Damage Localization

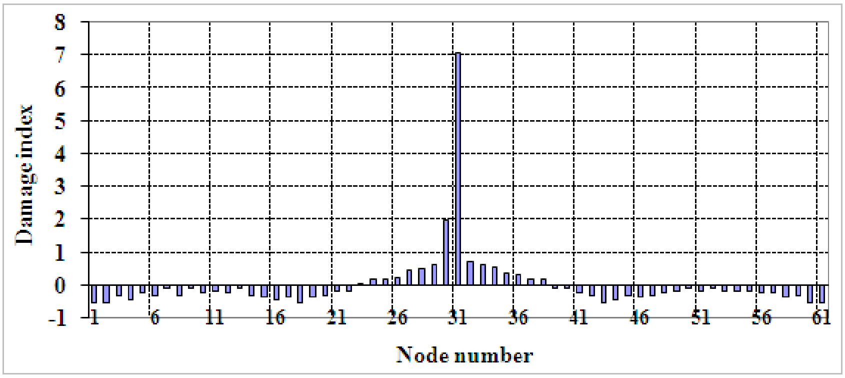
The damage indices of mode shape curvatures of *Case 4* of damage (10% reduction in *E* in element No. 30) are plotted in Fig. A.2 (a, b, c) for the first, second and fifth mode shapes, respectively. It is shown that the damage indices have clear spikes at the damaged element for each of the first five modes. The values of the damage indices beyond the damaged region have small values, (< 2), which represent the noise (measurements or approximation errors). However, there is a variation in the damage indices values from one mode to another. This depends on the sensitivity of each mode to a specific location of damage. So, the mode shape curvature for each of the first five mode shapes is able to localize structural damage, even with only the first mode shape.

The damage indices of mode shape curvatures of *Case 6* of damage (30%

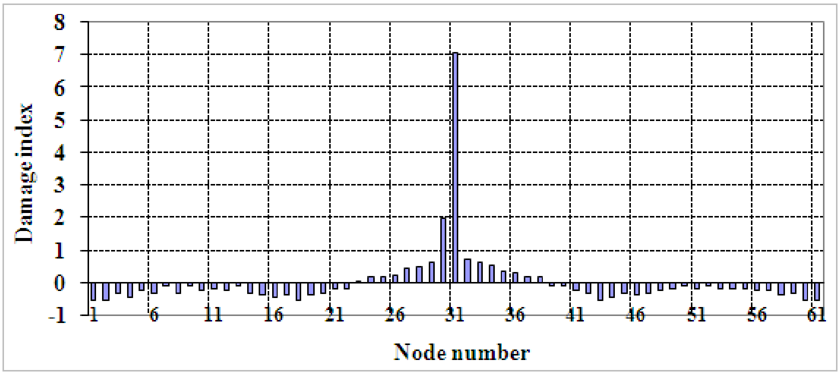
reduction in E in elements No. 1, 8, 15, and 30) are plotted in Fig. A.3, using only the first mode shape. It is shown that the damage indices of the curvature of mode shapes have clear spikes at the damaged elements of the beam model and have small values beyond the damaged elements. Indeed, although the method provides clear spikes at the four damage locations, the damage indices of element No. 1 are underestimated (< 2). This can be interpreted by the fact that the damage in element No. 1 is located in a region of zero bending moment and consequently zero curvature. Therefore, the curvatures are less sensitive to that location of damage compared with other damage locations where the curvature has a specific value. Fortunately, most of damage types are related to flexural failures, so the curvature techniques are promising in structural damage identification.



(a) First mode



(b) Second mode



(c) Fifth mode

Figure A.2 *Damage indices of mode shape curvature of Case 4 of damage.*

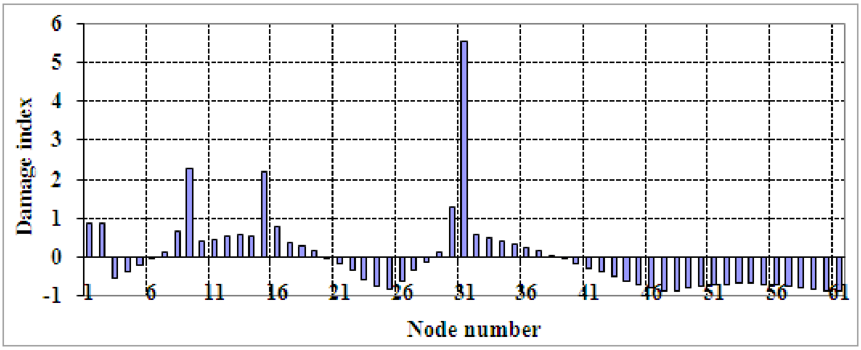


Figure A.3 *Damage indices of mode shape curvature of Case 6 of damage using the first mode shape.*

Appendix-B Damage Localization using Static Response Method

B.1 Grey Relation Coefficient (GRC)

In Section 2.4.2, it has been shown that the changes in displacement curvatures are sensitive and localized at the damaged region. So, we need to make a decision whether a structure is undamaged or damaged at a given location. In this section, the grey relation analysis is used as a damage localization algorithm.

Grey system theory was initiated by Deng (1989). Grey relation analysis, an essential part of the grey systems theory, deals with poor or incomplete data, or uncertain problems of some systems. It was successfully applied in many research fields. The grey relation coefficient (GRC) essentially indicates the approaching degree of two geometric curves: the larger the relational coefficient is, the nearer the geometric curves are, Chen *et al.*, (2005).

Now, if we have a reference sequence x_0 and test sequences x_i in the form: $x_0=(x_0(1), x_0(2), \ldots, x_0(j))$, $x_i=(x_i(1), x_i(2), \ldots, x_i(j))$, the expression of the GRC at point j, $\xi_i(j)$, is given by:

$$\xi_{\mathrm{i}}(j) = \frac{\min\limits_i \min\limits_j x + \alpha \max\limits_i \max\limits_j x}{x + \alpha \max\limits_i \max\limits_j x}. \tag{B.1}$$

where $x=|x_0(j)- x_i(j)|$; α is the distinguishable coefficient used to adjust the range of the comparison environment, and to control level of differences of the relation coefficients. The factor α usually ranges between 0.1 and 0.5, and in this study, α is taken 0.5 which is recommended by Deng, (1989). The grey relation coefficient $\xi_i(j)$, usually ranges from 0 to 1, evaluates the point-relation degree at the j th point of the test and reference sequences. Generally, $\xi_i(j)>0.9$ indicates that the reference point and the test point are related completely; $0.8<\xi_i(j)<0.9$ indicates a good relation of the two points; $0.6<\xi_i(j)<0.8$ indicates that the two points are relative or irrespective possibly; $\xi_i(j)<0.6$ represents that the two points are almost irrelative, Fu *et al.*, (2001). In this study, x_0 and x_i stand for undamaged and damaged measurements or analytical and measured vectors, respectively.

B.2 Some Results of Damage Localization

(1) Numerical Analysis

The model used in this analysis is the same one used in Appendix-A. A linear static analysis is performed to examine the robustness of using displacement curvature technique in damage detection. The beam is analyzed under uniform distributed load of 60 [kN/m].

Only the translational displacement in the y-direction (u_y) is considered in the analysis. This was done because, in any experimental work, in general, rotations are not measured because of the difficulty in their measurements. Also, since we are interested only in the flexural response, axial translation along the x axis can also be neglected. The curvature at each node is estimated using a finite central differentiation procedure. Using this technique, the displacement curvature at the ith node is calculated by a Laplacian operator along the beam length using Eq. (3.9). If two sets of measurements, one from the intact structure and another from the damaged structure are taken, the displacement curvature at any point for the two states can be obtained using Eq. (3.9). Then the GRC value is estimated at each node using Eq. (B.1) on the absolute difference of displacement curvatures for each case of study.

As mentioned above, the damage of the model in this study is assumed to affect the stiffness of an element and the change in the stiffness due to damage is modeled by a reduction in the Young's modulus of the element. The degree of damage is then related to the extent of reduction of the Young's modulus, *E*, of some elements. Five scenarios of damages are studied. These scenarios of damage are investigated to represent not only different locations but also different severities of damages. Table B.1 describes the damage characteristics of the five scenarios of damage. The first and second scenarios of damage represent single damage with 30% reduction in Young's modulus, *E*, with different locations in elements No. 1, 12, respectively. The third and fourth scenarios of damage represents single damage with different severity of damage, 10% and 30% reduction in Young's modulus of element 30. The fifth scenario of damage represents multiple damage locations with 30% reduction in Young's modulus in elements No. 1, 12, and 30, simultaneously. The three elements No. 1, 12, and 30 are chosen to simulate different damage types. Damage in element No. 1 represents a failure due to pure shear at the end support. Also, damage in element No. 12 represents a failure due to pure bending moment. However, damage in element No. 30 represents a failure due to combined shear and bending moment. Since the displacement curvature is estimated approximately from the measured displacement and inevitable errors are expected, the changes in displacement curvatures under uniform distributed load are assumed to be contaminated with random noise, which is inflicted as 5% and 10% of the root mean square of the original data, to simulate the measured error. More details of the model are found in Abdo (2012).

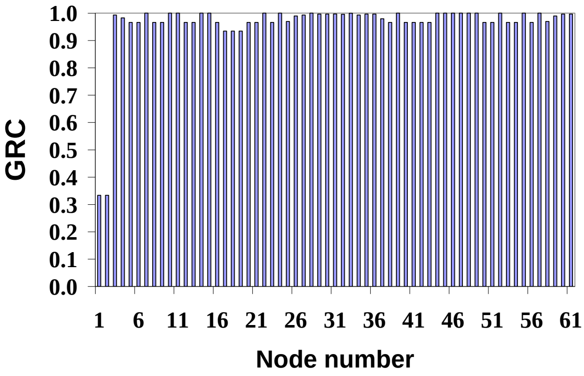
Table B.1 *Damage characteristics of the two-span beam model.*

Scenario of damage	**Damage location (Element No.)**	**Percentage reduction in E**	**Number of damage locations**
Scenario-1	1	30	1
Scenario-2	12	30	1
Scenario-3	30	10	1
Scenario-4	30	30	1
Scenario-5	1, 12, 30	30 in all elements	3

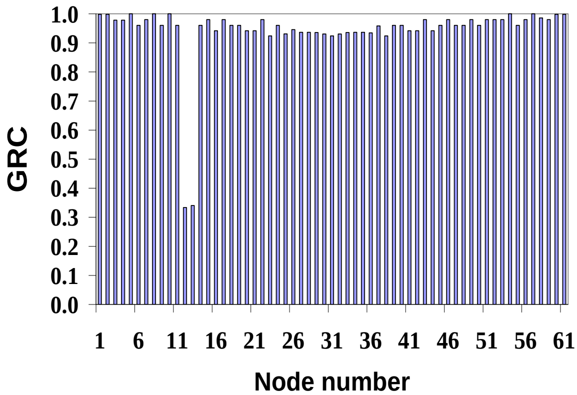
(2) Results of Damage Localization

The GRC values for the difference of displacement curvatures between the intact and the damaged beam under uniform distributed load for the first four scenarios of damages are plotted in Fig. B.1 (a to d). Also, The GRC values of displacement curvatures between the intact and the damaged beam under uniform distributed load for *scenario-5* of damage (30% reduction in Young's modulus of elements No. 1, 12, and 30) are plotted in Fig. B.2. It is shown that the GRC values are over 0.90 all over the beam length, but decrease greatly with a drop at the two nodes of the damaged elements. This indicates that the correlation of the distinguished elements is very poor, which means that there are damages at these elements. Therefore, the damage detection algorithm using changes in displacement curvatures can accurately locate different damage characteristics.

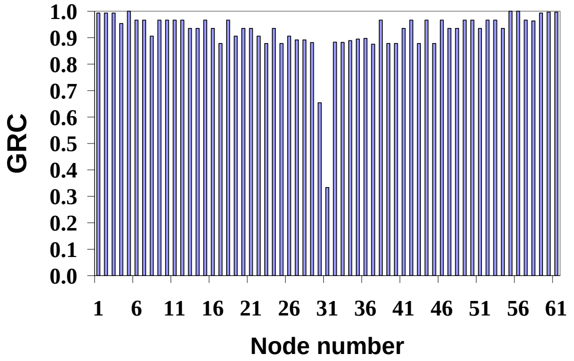
Furthermore, Figure B.3 (a, b) plots the GRC values for the changes in displacement curvatures which contaminated with a normal distribution measurement noise; 5% and 10%, respectively, for *scenario-5* (multiple damages). Again, it is shown that the GRC values are over 0.90 all over the beam length, but decrease greatly with a drop at the two nodes of the damaged elements. Thus, it is clear that even with 10% noise, spikes at the damaged nodes have occurred and the GRC values are in good agreement with those obtained with error-free measurements (Fig. B.2).



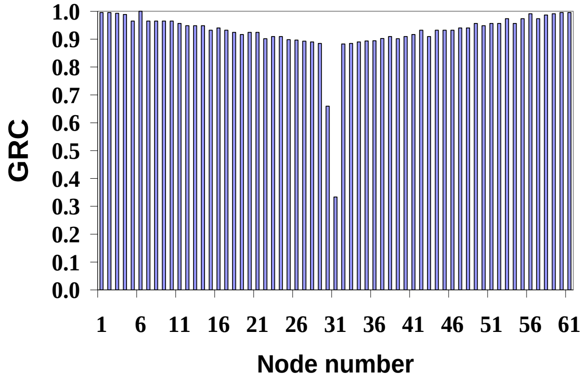
(a) Scenario-1



(b) Scenario-2



(c) Scenario-3



(d) Scenario-4

Figure B.1 *Damage localization of scenarios: 1, 2, 3, and 4 (single damage) for the two-span continuous beam.*

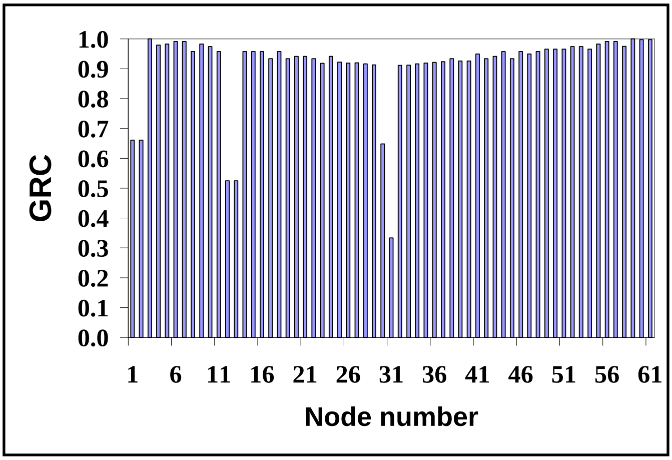
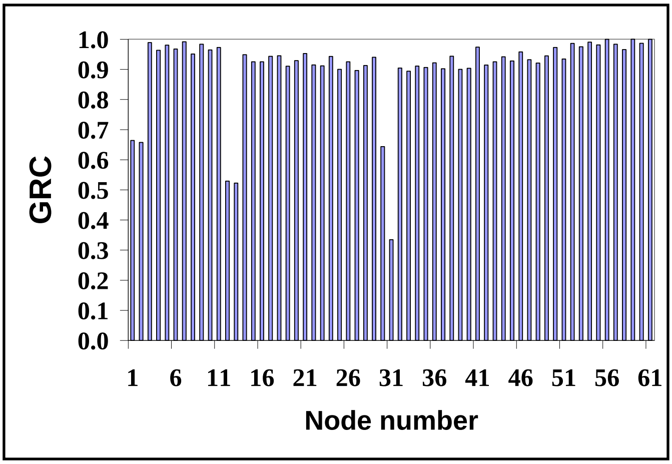
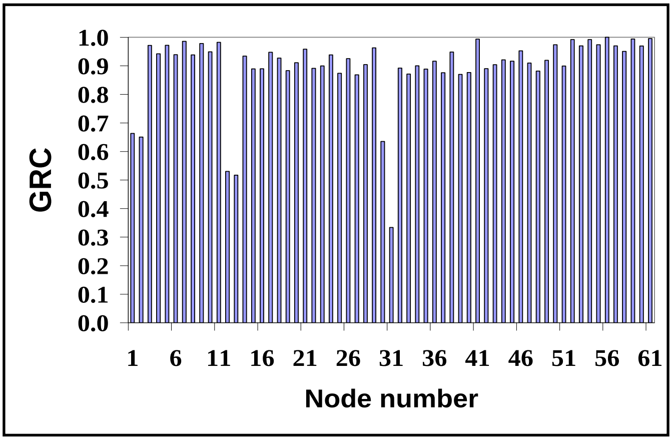


Figure B.2 *Damage localization of scenario-5 (multiple damages) for the two-span continuous beam.*



(a) 5% noise



(b) 10% noise

Figure B.3 *Damage localization of scenario-5 (multiple damages) for the two-span continuous beam with measurement noise.*

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