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HEALTH MONITORING AND VIBRATION CONTROL
OF STEEL SPACE STRUCTURES

By

CHEN Bo


A Thesis Submitted in Partial Fulfillment of the Requirements for the
Degree of Doctor of Philosophy

Department of Civil and Structural Engineering
The Hong Kong Polytechnic University
January 2007
To my family
DECLARATION

I hereby declare that this dissertation entitled "Health Monitoring and Vibration Control of Steel Space Structures" has not been, either in whole or in part, previously submitted for a degree in this or any other institution, and the work presented in this thesis is original unless otherwise acknowledged in the text.

SIGNED

__________________
CHEN Bo
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Abstract

This thesis pursues the understanding of structural behaviour of steel space structures under various types of external loads including atmospheric and stress corrosion, the development of innovative yet practical algorithm for structural damage detection, the combination of health monitoring with vibration control towards a smart steel space structure, and the formation of integrated structural health monitoring and vibration control systems for the best protection of steel space structures.

Steel space structures exposed to the open air are inevitably subjected to atmospheric corrosion. This thesis first presents a framework for evaluation of potential damage due to atmospheric corrosion to steel space structures through an integration of knowledge in material science and structural analysis. An empirical model for estimating corrosion of steel material is presented based on long-term experimental data available. Equations relating the sensitivity of structural natural frequencies to the thickness of structural members are derived in consideration of both inner and
outer surface corrosions of structural members. A nonlinear static analysis is conducted to evaluate effects of atmospheric corrosion on the stresses of structural members and the safety of steel space structures. By taking a large steel space structure and a reticulated steel shell as two examples, the feasibility of the proposed approach is examined and the potential damage caused by atmospheric corrosion to the structures is assessed. The results demonstrate that the atmospheric corrosion does not obviously affect the natural frequencies of the structures but it does create stress redistribution and cause large stress changes in some of the structural members.

The research work on atmospheric corrosion of steel space structures is then extended by involving stress corrosion cracking to estimate corrosion damage to steel space structures in a more realistic way. An evaluation method for coupled atmospheric corrosion and stress corrosion cracking of steel space structures is presented in consideration of different locations and shapes of initial cracks as well as different periods of atmospheric corrosion. The proposed method is applied to the large steel space structure to evaluate its potential corrosion damage. Based on the analytical results of atmospheric corrosion and stress corrosion cracking and the sensory technology, a corrosion monitoring system is conceptually designed to monitor the large steel space structure in corrosive environment and to update the proposed evaluation model, which will also form a sub-system of the integrated health monitoring and vibration control system for the reticulated steel shell in the last phase of this study.

The corrosion-induced fracture or local instability of a steel space structure may cause sudden stiffness reductions of some structural members, which will induce the
discontinuity in acceleration response time histories recorded in the vicinity of damage location at damage time instant. An instantaneous damage index is proposed to detect the damage time instant, location, and severity of structures due to a sudden change of structural stiffness. The proposed damage index is suitable for online structural health monitoring. It can also be used in conjunction with the empirical mode decomposition for damage detection without using intermittency check. A shear building and the reticulated shell are respectively selected to numerically assess the effectiveness and reliability of the proposed damage index with different types of excitation and different levels of damage being considered. The sensitivity of the damage index to the intensity and frequency range of measurement noise is also examined. The results demonstrate that the damage index and damage detection approach proposed can accurately identify the damage time instant and location in the structures due to a sudden loss of stiffness if measurement noise is below a certain level. The relation between the damage severity and the proposed damage index is linear.

In most of previous investigations, structural health monitoring and structural vibration control have been treated separately. This study presents an integrated procedure for health monitoring and vibration control of structures using semi-active friction dampers towards a smart structure. The concept of integrated health monitoring and vibration control systems using semi-active friction dampers is introduced by means of a shear building subject to earthquake excitation. It is then applied to the reticulated steel shell with some adjustments in control algorithm and system identification procedure. In such an integrated approach, a model updating scheme based on adding known stiffness by using semi-active friction dampers is
first presented to update the structural stiffness and mass matrices and to identify its structural parameters using measured modal information. Based on the updated system matrices, the control performance of semi-active friction dampers with a given control algorithm is then investigated for either the building or the shell against earthquakes. By assuming that the building or the shell suffers certain damage after an extreme event or long-term service and by using the previously identified original structural parameters, a damage detection scheme based on adding known stiffness using semi-active friction dampers is proposed and used for damage detection. The feasibility and effectiveness of the proposed integrated procedure are demonstrated through detailed numerical investigation on the shear building and the reticulated shell.

For control devices which cannot provide the required two states of additional stiffness to a structure like the semi-active friction dampers, the parameter identification and damage detection of the controlled structure can be performed in the time domain as long as the control forces can be measured. The equation of motion of the controlled structure is first converted to the parametric identification equation when the inertia forces, damping forces, and restoring forces are linear functions of structural parameters. By taking control forces as known external forces together with measured structural responses, the least-squares method together with an amplitude-selective filter is then used to solve the parametric identification equation, from which the structural parameters can be identified. The same procedure is applied to the controlled structure with damage to identify another set of structural parameters. By comparing the two sets of structural parameters identified, the structural damage can finally be detected and quantified. This proposed procedure is
applied to the shear building and the reticulated steel shell with control devices for parametric identification and damage detection with and without measurement noise. The numerical results demonstrate the feasibility and effectiveness of the procedure when the measurement noise is small.

The conceptual design of an integrated health monitoring and vibration control system is finally performed in this thesis by taking the reticulated steel shell as an example with the aim of updating analytical models, identifying structural parameters, assessing structural safety, guiding maintenance and repairing work, and activating control devices to protect the structure against extreme loading. In this regard, the structural behaviour, stability and safety of the reticulated steel shell under dead load, wind load, earthquake load, temperature, fire and corrosion are investigated or summarised. Based on these understandings, various types of sensors are selected to measure climate change, atmospheric contamination, material corrosion, wind, earthquake, structural responses, and control forces among others. The numbers and locations of the sensors and control devices are also specified. Two databases are established to collect the information from the sensors and the inspection respectively. The main objectives of installing the integrated system are demonstrated based on the information collected and the layout of the integrated system is illustrated in detail.
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CHAPTER 1

INTRODUCTION

1.1 Motivation

The rapid economic growth in China is tremendously driving it towards greater openness, internationalization and civilization. The Olympic Games to be held in Beijing in 2008 will inevitably accelerate this process to a great extent and improve Chinese international influence across the world. To meet the requirements of Olympic Games, many large steel space structures including thirty-seven newly-established gymnasiums and stadiums have been designed and constructed in recent years. Most of these large-scale facilities are developed with large steel roof to provide enough spaces for various sporting competitions (Liu 2005). The 332.3 m long and 297.3 m wide National Stadium located in Beijing for the opening and closing ceremony is fully constructed with steel components and has the capacity to hold about 100,000 people. The National Swimming Centre (also named Water Cube) is a large steel space frame with 170 m in length, 170 m in width and 29 m in height. The National Gymnasium is constructed with a large steel space roof with horizontal dimensions of 114 m in length and 144.5 m in width. The Olympic Badminton Gymnasium is developed as a steel reticulated shell with a span of 105 m and a height of 32.43 m. Many other important steel space structures are also constructed for the coming Olympic Games such as the Olympic Bicycle Gymnasium, the Olympic Basketball Gymnasium, the Olympic Table Tennis Gymnasium, the Olympic Rassling Gymnasium and the Olympic Stadium in Tianjin. In addition, to meet the growing needs for social and economic development, many other large steel space...
structures such as exhibition centers and airports are also built especially in those coastal metropolises with flourishing economy. The design and construction of large steel space structures with special configurations and functions present many new challenges to civil engineers and designers because these steel structures are always subjected to harsh environment, such as corrosion, vibration, fatigue, material aging and various external loads, which may lead to structural damage events.

The damage events of steel space structures may occur in harsh environment and intensive external loads during long-term service. If the accumulated damage cannot be detected timely, the structural safety will be threatened and the damage may finally cause partial or even total collapse of the structure, resulting in huge economic loss and fatal casualty. A vast reticulated shell in Bucharest, Romania collapsed in 1963 only 17 months after its erection at a low exterior temperature and during a snowstorm which was caused by a loss of elastic stability due to the locally accumulated snow loads (Soare 1963; Beles et al., 1967). Similarly, the space truss roof of the Hartford Coliseum in Connecticut, USA also collapsed in 1978 after a strong snowstorm. The further examination revealed that the shell failure was caused by the lack of force bearing capacity under extreme large snow loads (Smith et al., 1980). In addition, many failures of steel space structures have been reported in China in recent years (Lei 2003). The steel roof of Fengyuan High School in Taiwan suddenly collapsed in 1977 which killed 26 people and seriously injured 30 plus. This accident was caused by the long-term overloading and corrosion of steel members. The steel roof of the Shenzhen International Exhibition Center collapsed in 1994 due to the nodal damages and extreme rain loads. A cable-suspended steel roof in Shanghai suddenly collapsed after twenty years of service because of corrosion-
induced fracture of steel components. The space roof of the Paris' Charles de Gaulle International Airport is a composite space structure consisting of steel frames and a concrete shell. The partial roof of the departure area at Terminal 2E of this airport collapsed in May 23, 2004, a little more than two years after it was built, killing 4 people and injuring other 3. The investigation report provided by an independent commission pointed out several major reasons for the failure: (1) insufficient or badly positioned steel components; (2) lack of mechanical redundancy for structural components because of stress concentration; and (3) major beams that offered too little resistance to stress. During the long-term service, the space structures may be subjected to various external loads and extreme events such as dead loads, earthquake, wind, temperature effects, instability, fire and corrosion among others. Thus, reasonable measures should be taken to reduce the structural responses under intensive external loads and monitor the potential structural damages which will be a very challenging issue to be assessed.

The steel space structures, different to other space structures such as long span concrete structures, are constructed fully using steel material which is prone to corrosion damage in corrosive environment. As a typical corrosion type for metal material, the atmospheric corrosion is widely observed for engineering structures. Atmospheric corrosion can be defined as the corrosion of materials exposed to air and its contaminants rather than immersed in a liquid (Roberge 2000). International concern has also increased over the past decade as it has become evident that atmospheric corrosion has resulted in substantial deterioration of buildings and structures (Cowell and Apsimon, 1996; Ninomiya et al., 1997). The influence of atmospheric corrosion on reinforcing steel is effectively investigated (Ibrahim et al.,
1994; Batis and Rakanta 2004) to evaluate the steel weight loss, strength, elongation and bending ability. The safety evaluation on small steel facility under atmospheric corrosion such as steel post is also executed to reveal damage states (Herrera et al., 1995). A little work has yet been carried out to evaluate effects of atmospheric corrosion on structural behavior and safety of large steel space structures especially built in coastal areas.

Apart from atmospheric corrosion, the stress corrosion cracking (SCC) and corrosion fatigue are two other corrosion damages which are normally observed for steel structures under the influence of corrosive environment, external loadings and cracks. SCC usually refers to the failure of components due to crack propagation of members under static loads, while environmentally induced crack propagation under cyclic loads is normally defined as corrosion fatigue (Talbot et al., 1998). The researches on SCC and corrosion fatigues have been carried out for many years and several analytical models and techniques were developed to localize and evaluate corrosion damage. Current researches on corrosion damage of civil engineering structures, however, mainly focus on the structural components rather than the whole structure. No works have been carried out to effectively evaluate the structural performance under the interaction of different corrosion damages such as atmospheric corrosion and SCC. Moreover, the corrosion monitoring strategy and system for steel space structures have also not been systematically developed for application.

The accumulation of corrosion damage in the structural components with the increase in service duration may finally cause fracture, resulting in partial or whole destruction of structural members. In addition, the steel space structures may undergo
instability if they work under strong external loads. The corrosion-induced fracture or structural instability may cause the sudden stiffness reduction of structural members which may further cause a discontinuity in acceleration response time histories recorded in the vicinity of damage location at damage time instant. Many damage indices based on wavelet transform (WT) and empirical mode decomposition (EMD) were theoretically and experimentally developed to acquire a damage feature retaining damage time instant (Hou et al., 1999, 2000; Yang et al., 2001, 2004; Xu and Chen 2004). However, both the numerical study and the experimental investigation demonstrate that the relationship between damage spike amplitude and damage severity could not be given by either the WT or the EMD with intermittency check. To this end, Yang et al. (2004) suggested an alternative method based on the EMD with intermittency check and Hilbert transform to quantitatively detect the damage time instant and the natural frequencies and damping ratios of the structure before and after damage. However, this multi-stage method proposed by Yang et al. (2004) may not be suitable for online structural health monitoring applications. How to develop instantaneous approach for detecting the sudden damage events and reflecting the damage extent for online health monitoring should thus be investigated.

Besides the sudden damage events, the slow damage events of steel space structures may be caused by various reasons such as operating loads, earthquake, wind, corrosion, temperature change, fire, etc., in general, producing changes in the structural physical properties (i.e. stiffness, mass and damping), and these changes will lead to changes in the dynamic characteristics or dynamic responses of the structures. This fact has been widely noticed and used by structural engineers for
damage detection or health monitoring (Dowbling 1996, 1998). The reasonable approaches should be proposed to identify the slow damage events of steel space structures.

To protect the structures against intensive external excitations such as earthquake and provide enough safety provisions, vibration control techniques are adopted for various civil engineering structures. Passive control technology has been evolved into a workable technology and implemented in real structures but its control performance is always limited because of its passive nature. Active/hybrid control technology has also been implemented in several structures but cost effectiveness and reliability consideration have limited their wide acceptance in civil engineering. Semi-active control technology is now receiving considerable attention from engineering professionals because it offers the reliability of passive control systems and at the same time maintains the versatility and adaptability of active/hybrid control systems with much lower power requirement which is a critical issue during earthquakes for operating active/hybrid control systems. Although the theoretical and experimental researches on semi-active control of civil engineering structures have been conducted for many years, the control approaches for steel space structures have not been effectively developed up to now. Detailed investigation is needed.

To protect the structures against strong external excitations and extreme events, monitoring and control devices can be installed. Vibration control and health monitoring of civil engineering structures under harsh environment have been actively investigated in recent years. In the aspect of vibration control, several technologies have been developed for reducing the excessive vibration of civil
engineering structures caused by strong winds, severe earthquakes or other disturbances. In the aspect of health monitoring, structural monitoring systems consisting of sensors, data acquisition and data analysis have been developed and implemented in civil engineering structures for the identification of dynamic characteristics and parameters and for the detection of possible damage after extreme event or long-term service. Although both vibration control system and health monitoring system need sensors and data acquisition and transmission, vibration control and health monitoring have been treated separately in most of the investigations according to the primary objective pursued. This separate approach is not practical and cost-effective if structures do require both vibration control system and health monitoring system. This separate approach is also not beneficial for the utmost goal of creating smart structures with their own sensors (nervous system), processor (brain system), and actuators (muscular system)-thus mimicking biological systems. In this regard, Ray and Tian (1999) proposed a method of enhancing modal sensitivity to local damage using feedback control to aid in damage detection. Gattulli and Romeo (2000) proposed the use of an integrated procedure for robust control of oscillations and damage detection of linear structural systems. Sun and Tong (2003) presented a closed-loop control based damage detection scheme aiming at detecting small damage in controlled structures. These studies, however, focus on actively controlled mechanical systems or small structures. For large-scale civil engineering structures, such as steel space structures, it becomes necessary to investigate the development of an integrated procedure for health monitoring and vibration control of steel space structures for the evaluation of system safety and vibration mitigation.
1.2 Objectives

This thesis focuses on the understanding of structural behaviour of steel space structures under various types of external loads including corrosion, the development of innovative yet practical algorithm for damage detection, and the integration of health monitoring and vibration control techniques for the best protection of steel space structures. The major objectives of this research are as follows:

1. To introduce a framework for the evaluation of potential damage due to atmospheric corrosion to steel space structures. The refined exponential model will be developed to estimate atmospheric corrosion of steel materials at a site. Based on the refined prediction model, an additional analytical study will be performed to determine the key parameters in the prediction model. The assessment on the sensitivity of natural frequency to variation of member thickness due to atmospheric corrosion is important for safety evaluation. The nonlinear static structural analysis will be conducted to evaluate the effects of atmospheric corrosion on the stress of structural members and the safety of steel space structures. The feasibility of the proposed approach on evaluating atmospheric corrosion damage of the structure should be examined through a real steel space structure.

2. To extend the research work on atmospheric corrosion damage of steel space structure by involving the SCC evaluation to provide an integrated procedure for evaluation techniques and monitoring approach on corrosion damages of steel space structures. The analytical procedure of member SCC will be presented by involving the influence of atmospheric corrosion. The influence of atmospheric
corrosion on SCC is an important aspect required to be investigated to examine the interaction of these two corrosion damages. The feasibility and effectiveness of the proposed evaluation approach of corrosion damages will be assessed through the example steel space structure to examine the structural safety. The methodology for designing a corrosion monitoring system is another objective of this research.

3. To develop an instantaneous index for sudden damage events based on the signal discontinuities in the acceleration response time histories caused by sudden damage event. The features of signal discontinuities in the acceleration response time histories recorded in the vicinity of damage location due to a sudden damage event should be examined for proposing an instantaneous damage index to detect the damage time instant and location. The relationship between index magnitudes and damage severity should be explored to detect the damage extent. The damage detection under sinusoidal, impulse and seismic excitations will be conducted respectively to understand the detection performance of proposed damage index. The effects of noise contamination and damage severity on detection performance under various excitations are also investigated to examine the detection robustness.

4. To present an integrated procedure for health monitoring and vibration control of steel space structures using semi-active friction dampers. The concept of integrated health monitoring and vibration control systems using semi-active friction dampers needs to be introduced. The model updating schemes by using semi-active friction dampers are developed in both frequency and time domains.
to update the structural stiffness and mass matrices and to identify structural parameters. Within the semi-active control, the differences in control performance provided by the local- and global-feedback controllers are an aspect required to be verified for assessment on control performance. The detection scheme by using semi-active friction dampers will be proposed and used for damage detection. The influence of noise contamination on model updating, vibration control and damage detection is required to examine the identification and control performance. The feasibility and accuracy of the proposed integrated procedure should be assessed through detailed numerical examples and parameter studies.

5. To design the integrated health monitoring and vibration control system for steel space structures by involving the knowledge in corrosion science, control theories, structural analysis and sensory technology for operation and maintenance of other steel space structures to be built. The conceptual system layout and operation of the integrated control and monitoring system should be illustrated. The type, amount and distribution of various sensors will be determined to achieve the goals of monitoring climate change, atmospheric contaminants, material corrosion, structural responses and working states of control devices to execute the health monitoring and vibration control for the best protection of steel space structures.
1.3 ASSUMPTIONS AND LIMITATIONS

The development and application of health monitoring and vibration control techniques for steel space structures investigated in this thesis are subject to the following assumptions and limitations:

1. A real reticulated shell erected in China is taken as a major example throughout this thesis to demonstrate the integrated health monitoring and vibration control in a systematic way. Nevertheless, simple shear building is used as an additional example in some chapters to facilitate the understanding of the basic concept of integrated health monitoring and vibration control and its application to building structures. Furthermore, a real complex space structure is also utilized to demonstrate the feasibility of the corrosion damage evaluation procedure in addition to the reticulated shell.

2. Because of the limitation of time and facility, wind tunnel tests cannot be performed in this study to obtain wind pressure time histories for wind-induced vibration analysis of steel space structures. Only empirical methods are used in this study for wind-induced vibration analysis of space structures.

3. The evaluation of corrosion fatigue damage is not executed in this study because wind-induced pressure time histories are not available and the reticulated shell is dominated mainly by compressive forces. The load combination as specified in some design codes is also not taken into consideration since this study aims to have a full understanding of individual load effects for integrated health monitoring and vibration control.
4. In the dynamic analysis, the space structures are assumed to vibrate within the elastic linear range. However, geometric and material nonlinearities are considered in the static analysis and thermal analysis.

5. The semi-active friction damper is used in this study as a major component of an integrated health monitoring and vibration control system. The study is numerical only, and no experimental work is attempted to verify the numerical results because of time limitation.

6. The conceptual design of the integrated health monitoring and vibration control system is performed in this thesis by taking the reticulated steel shell as an example to lay down a useful framework. The real implementation of the integrated system in the reticulated shell is beyond this study.

1.4 THESIS LAYOUT

This thesis contains a number of research topics to achieve the aforementioned objectives. It is divided into ten chapters and is organized in the following way.

Chapter 1 introduces the motivation of the present study and clearly states the objectives to be pursued and the assumptions and limitations adopted in this research study.

Chapter 2 contains an extensive literature review on relevant topics, which covers five subjects. The five subjects include atmospheric corrosion, SCC, structural
control, damage detection and health monitoring. Atmospheric corrosion of civil engineering structures is reviewed under the following topics: mechanism, prediction model and its effects on the structures. Then reviews are placed on the research studies and applications of SCC on civil engineering structures in which three are briefly introduced: crack expansion in corrosive environment, sensor and monitoring techniques, evaluation methods of corrosion damage. After this, structural control technology is reviewed to introduce the characteristics of various structural control approaches and major attentions are paid to the advances of variable friction devices. The development of system identification and damage detection technology is extensively reviewed under the following classifications: index methods, model updating methods, signal based methods and regularization techniques. The development of health monitoring technology is finally reviewed which includes the introduction of monitoring procedure, sensor technology and its application and limitations.

Chapter 3 presents the refined exponential model for estimating corrosion of steel materials at a site using a pattern recognition technique to determine the key parameters in the model. The formulae for relating structural natural frequency sensitivity to structural member thickness are then derived to assess the sensitivity of natural frequency to variation of member thickness due to atmospheric corrosion. The nonlinear static structural analysis is finally conducted to evaluate effects of atmospheric corrosion on the stresses of structural members and the safety of steel space structures. A large steel space structure and a reticulated shell built in China are taken as the case study to examine the feasibility of the proposed approach and to assess the potential damage caused by atmospheric corrosion to the structure.
Chapter 4 extends the research work on atmospheric corrosion of steel space structures by involving SCC to estimate corrosion damage to steel space structures in a more realistic way. An evaluation method for coupled atmospheric corrosion and SCC of steel space structures is presented taking into consideration the different locations and shapes of initial cracks as well as different periods of atmospheric corrosion. The proposed method is applied to the large steel space structure to evaluate its potential corrosion damage. Based on the analytical results of atmospheric corrosion and SCC and the sensory technology, a corrosion monitoring system is conceptually designed to monitor the large steel space structure in corrosive environment and to update the proposed evaluation model. The corrosion monitoring system will also form a sub-system of the integrated health monitoring and vibration control system for the reticulated steel shell in the last phase of this study.

Chapter 5 conducts the detection of sudden stiffness reduction caused by corrosion-induced fracture or local instability of some structural members, which will induce the discontinuity in acceleration response time histories recorded in the vicinity of damage location at damage time instant. A new instantaneous damage index is then proposed to detect the damage time instant, location, and severity of a structure due to a sudden change in stiffness. The proposed damage index is proportional to damage severity and suitable for online structural health monitoring applications. It can also be used in conjunction with the EMD for damage detection without using the intermittency check. Numerical simulation is executed to assess the effectiveness and reliability of the proposed damage detection approach for different types of
excitation and at different levels of damage. The sensitivity of the damage index to the intensity and frequency range of measurement noise is also examined. The damage detection results from the proposed approaches are also compared with those from the WT.

Chapter 6 presents an integrated procedure for health monitoring and vibration control of building structures using semi-active friction dampers. The concept of integrated system using semi-active friction dampers is firstly introduced by means of a shear building subjected to earthquake excitation. A model updating scheme based on adding known stiffness by using semi-active friction dampers is then presented to update the stiffness and mass matrices of a building and to identify its structural parameters. Based on the updated system matrices, the control performance of semi-active friction dampers using local feedback control with a Kalman filter is investigated under earthquakes. By assuming that the building suffers certain damage after extreme event or long-term service and by using the previously identified original structural parameters, a damage detection scheme based on adding known stiffness using semi-active friction dampers is finally proposed and used for damage detection. The feasibility and accuracy of the proposed integrated procedure are demonstrated through detailed numerical examples and parameter studies which include the effects of measurement noise, incomplete measurement information, damper (brace) stiffness and seismic excitations.

Chapter 7 verifies the application of the proposed integrated health monitoring and vibration control procedure to the reticulated shell with some adjustments in control
algorithm and system identification procedure. A practical approach for incorporating semi-active friction damper into the reticulated shell is presented. Based on the dynamic responses of the reticulated shell under earthquakes, two damper installation schemes are suggested for seismic mitigation. The equation of motion of the reticulated shell subjected to earthquakes and control forces is deduced and a local control strategy only using the local information of semi-active friction dampers is utilized to realize the seismic mitigation. The control performance of two damper installation schemes is compared in order to determine the most appropriate damper position for satisfactory control performance. The model updating scheme based on adding known stiffness proposed in Chapter 6 is extended for the reticulated shell. The transform matrix of stiffness parameters are deduced based on the structural connectivity and transformation information. By assuming that the shell suffers certain damage after an extreme event or long-term service and by using the previously identified original structural parameters, the damage detection based on adding known stiffness using semi-active friction dampers is executed. The feasibility and effectiveness of the proposed integrated procedure are demonstrated through detailed numerical investigation without/with noise contamination.

Chapter 8 studies the feasibility of applying the integrated monitoring and control system to conduct system identification in time domain. For control devices which cannot provide the required two states of additional stiffness to a structure like the semi-active friction dampers, the parameter identification and damage detection of the controlled structure can be performed in the time domain as long as the control forces can be measured. The equation of motion of the controlled structure is first converted to the parametric identification equation when the inertia forces, damping
forces, and restoring forces are linear functions of structural parameters. By taking control forces as known external forces together with measured structural responses, the least-squares method together with an amplitude-selective filter is then used to solve the parametric identification equation, from which the structural parameters can be identified. The same procedure is applied to the controlled structure with damage to identify another set of structural parameters. By comparing the two sets of structural parameters identified, the structural damage can finally be detected and quantified. This proposed procedure is applied to the shear building and the reticulated steel shell with control devices for parametric identification and damage detection with and without measurement noise.

Chapter 9 performs the conceptual design of an integrated health monitoring and vibration control system by taking the reticulated shell as an example with the aim of updating analytical models, identifying structural parameters, assessing structural safety, guiding maintenance and repairing work, and activating control devices to protect the structure against extreme loading. In this regard, the structural behaviour, stability and safety of the reticulated steel shell under dead load, wind load, earthquake load, temperature, fire and corrosion are investigated or summarised. Based on these understandings, various types of sensors are selected to measure climate change, atmospheric contamination, material corrosion, wind, earthquake, structural responses, and control forces among others. The numbers and locations of the sensors and control devices are also specified. Two databases are established to collect the information from the sensors and the inspection respectively. The main objectives of installing the integrated system are demonstrated based on the information collected and the layout of the integrated system is illustrated in detail.
Finally, Chapter 10 summarizes the key results, draws conclusions observed in this study and provides some recommendations for further studies on this topic.
As mentioned in Chapter 1, this thesis pursues the understanding of structural behaviour of steel space structures under various types of external loads including corrosion, the development of innovative yet practical algorithm for damage detection, and the integration of vibration control and health monitoring techniques for the best protection of steel space structures. The research progresses achieved on vibration control and health monitoring have been reviewed in this chapter with focuses on atmospheric corrosion, stress corrosion cracking, vibration control, system identification, damage detection, and health monitoring.

2.1 ATMOSPHERIC CORROSION

2.1.1 Mechanism of atmospheric corrosion

The atmospheric corrosion is a normal phenomenon resulting from the chemical or electrochemical action between humid environmental and metal material exposed in the open air (Roberge 2000; Revie 2000). The research work up to now indicates that there are many factors such as climate, atmospheric contaminants, metal surface condition, etc can affect the extent of metal atmospheric corrosion (Talbot et al., 1998). Climatic conditions include humidity, temperature, sunshine and other climatic factors. Temperature can affect metal atmospheric corrosion because the variation of temperature leads to the change in atmospheric humidity. In high relative humidity environment, the decreasing temperature causes the vapor temperature
lower than the dew point, which will make the metal surface humid and further accelerate the atmospheric corrosion process. Under natural conditions, metal atmospheric corrosion is normally electrochemical corrosion. The effects of relative humidity on atmospheric corrosion are nonlinear and there exists a relative humidity value called critical humidity, beyond which the metal corrosion velocity increases significantly. The presence of atmospheric contaminants such as SO\textsubscript{2}, NO\textsubscript{2}, H\textsubscript{2}S, etc will substantially affect the extent of metal atmospheric corrosion.

2.1.2 Prediction model for atmospheric corrosion

The past four decades witness the rapid development of atmospheric corrosion research in the field of material science with many field measurements and corrosion evaluation methods being proposed (Kucera, 1986; Shastry et al., 1988). For instance, the Chinese Science and Technology Committee and the National Natural Scientific Foundation have established a national experimental network for atmospheric corrosion of metal materials since 1983, and a huge amount of data have been collected since then (Wang et al., 1995). Similar activities have also been carried out in other parts of the world (Talbot et al., 1998). Based on the data collected in field measurement, several empirical models have been put forward to predict the atmospheric corrosion of metal materials (Cole, 1994; Farrow and Graedel, 1996; Juan, 2003; Hou and Liang, 2004).

Feliu et al (1993) statistically compiled worldwide atmospheric corrosion and environmental data to establish general corrosion damage functions for mild steel, zinc, copper and aluminium, in terms of simple meteorological and pollution parameters. Their research proves that long-term atmospheric corrosion forecasts can
be fulfilled following the exponential model $D=AT^n$ with appropriate values of constants $A$ and $n$. They also explored the possibility of expressing $A$ as a function of commonly available environmental parameters, and the data compiled in a comprehensive literature survey are used to determine whether the exponent $n$ of the above equation can also be expressed as a function of such environmental parameters. Wang et al (1995) put forward the concept of the comprehensive factor $N$ to quantitatively describe the atmospheric corrosion of the metal material in the first year. The comprehensive factor $N$ consists of humidity coefficient, air contamination coefficient and rain acidity coefficient, which reflects the effects of environmental factors such as atmospheric contaminants, humidity, sunshine, and precipitation. The mathematical relationship between $N$ and the corrosion depth of carbon or low alloy steels for the first year is established and proved to be effective in accordance with the experimental data obtained in China. Hou and Liang (1994, 1998) examined the atmospheric corrosion data of steels after a 16-year exposure at various sites in China and used the power function to regressively analyze the loss of mass induced by corrosion. In their analysis, chemical composition of steel and environment factors is used as the arguments in the step regression. Quantitative relationship among the corrosion extent, environment factors and chemical compositions of steels were obtained. They demonstrated that the atmospheric corrosion can be predicted for most carbon and low-alloy steels. Most of the currently proposed prediction models based on experimental data are regressive models and they have been proved to effectively capture the nature of the atmospheric corrosion process. Among these models, the exponential model is mostly accepted due to its simplicity and satisfactory compliance with experimental data.
2.1.3 Influence of atmospheric corrosion on civil engineering structures

International concern has also increased over the past decade as it has become evident that atmospheric corrosion has resulted in substantial deterioration of buildings and structures (Cowell and Apsimon, 1996; Ninomiya et al., 1997). Ibrahim et al (1994) investigated the atmospheric corrosion of reinforcing steel and its influence on loss of steel mass, strength, elongation and bending ability. Batis and Rakanta (2004) examined the performance of four different sets of reinforcing steel bars exposed to the Greek atmosphere before their installation into the concrete. Herrera et al (1995) conducted an investigation on collapse of a 10m high steel post caused by atmospheric corrosion in USA. Flavio and Stefano (2002) carried out an extensive failure analysis trying to understand the reasons of the early failures of weathering steel used for many years for steel construction in the field of transportation. A little work has yet been carried out to evaluate effects of atmospheric corrosion on structural behavior and safety of steel space structures. The corresponding relationship between frequency change and the loss of section induced by atmospheric corrosion are not well understood. Furthermore, how to effectively evaluate the effects of atmospheric corrosion on the stress of structural members and the safety of steel space structures is still under investigation.

2.2 STRESS CORROSION CRACKING

The atmospheric corrosion of construction material may occur only under the corrosive atmospheric environment while the stress corrosion cracking (SCC) is commonly observed under the influence of corrosive environment, member stresses and cracks (Talbot et al 1998). SCC usually is described as brittle failure due to crack
propagation of structural components under static loads and corrosive environment. Damages induced by SCC are observed in various engineering structures such as boilers, pressure vessels, buildings, bridges, pipelines, ships, aircrafts, and marine infrastructures.

2.2.1 Crack expansion in corrosive environment

Failures in engineering structures occur at components or connections, even in those structures, which have been designed, fabricated, and inspected according to codes and specifications. Cracks in bridges, offshore structures, pressure vessels, and buildings always occur at the welded or bolted connections and attachments such as cover plate fillet weld terminations, stiffeners, backing bars, and girth weld toes. Various weld discontinuities, cracks, and crack-like imperfections have been widely reported (Barsom 1999). They are normally caused by various factors such as improper design, incorrect selection of a welding process or welding parameters and improper care of the electrode or flux. The typical flaws around welding connections are displayed in Figure 2.1. Normally, the severity of a crack is governed by its size, shape, orientation, and by the magnitude and direction of the design and fabrication stresses. In addition, the corrosive environment may accelerate the expansion of cracks. Codes and specifications define acceptance levels for cracks in terms of their type, size, orientation, and distribution. Commonly, cracks are prohibited when their size, orientation, and distribution exceed specification limits and their presence affects the integrity of the components to protect the individual components and the whole structures. Under the influence of corrosive environment and material stress, the crack may expand and the damages caused by SCC occur to threat the structural safety.
SCC is a phenomenon in which crack growth occurs when the necessary electrochemical and mechanical conditions exist (Russell 1992; Roberge 2000; Revie 2000). The observed crack propagation is the result of the combined interaction of mechanical stress and corrosive environment. The stresses required to cause SCC are usually small and tensile in nature. Static load is usually considered to cause SCC, while environmentally induced crack propagation under cyclic load is normally defined as corrosion fatigue. Russell (1992) and Roberge (2000) gave comprehensive reviews on crack expansion in corrosive environment and some observations from his work formed part of my literature review in this section. Following the illustration in his book, the sequence of events involved in the SCC process is usually divided into three stages, crack initiation, crack propagation and fracture. The magnitude of the stress distribution at the crack tip (the mechanical driving force for crack propagation) is termed as stress intensity factor (SIF) $K$. Figure 2.2 displays the variation of crack propagation rate with stress intensity factor (Russell 1992; Roberge 2000; Revie 2000). It can be found from Figure 2.2 that no crack propagation is observed below a threshold SIF $K_{ISC}$ and this threshold stress intensity is determined not only by the material type but also by the environmental condition around the material. If the SIF exceeds the threshold value, the crack propagates under the interaction of mechanical driving force and corrosive environment. At low stress intensity level (stage 1), the crack propagation rate increases rapidly with SIF. At intermediate stress intensity levels (stage 2), the crack propagation rate approaches some constant velocity that is virtually independent of the mechanical driving force. This plateau velocity is characteristic of the metal/environment combination. In stage 3, the crack propagation rate exceeds the
plateau velocity and the stress intensity level rapidly increases to exceed the critical SIF ($K_{IC}$) and cause final fracture.

2.2.2 Sensory and monitoring techniques

Various sensory and monitoring techniques addressing corrosion damages have been developed in recent years such as galvanic sensor, fiber optic sensor, ultrasonic method, etc. Short et al (1994) described the construction and operation of a galvanic sensor which responds to changes in the corrosion intensity of mild steel in carbonated mortars or concretes. The sensor is simple to be constructed and has potential for development as an inexpensive method for long-term, continuous non-destructive corrosion monitoring of steel in concrete. Mendoza et al (1998) reported distributed fiber optic sensors for use in the prevention of catastrophic corrosion failure when embedded in key structures such as high pressure gas and hazardous fluid pipeline delivery systems, potable water distribution piping, steel reinforced concrete structures, and steel and aluminum structures. Kelly et al (1999) designed a micro instrument for corrosion monitoring in reinforced concrete and discussed performance of the prototype device. Sensors for the measurement of corrosion rate, chloride concentration, and concrete conductivity are developed and tested inside of model concrete slabs. Simmonds et al (2000) introduced ultrasonic method for monitoring corrosion damage in petroleum metal storage tanks using spectral tracking. Initial research using this method demonstrates promising results in the possibility of mapping corrosion damage in the presence of sedimentation. Ralph and Juergen (2000) illustrated the corrosion monitoring sensors for durability assessment of concrete structures. The observations combined with other results contribute to the improved evaluation procedures for deterioration of concrete structures. Raupach et
al (2001) established macrocell sensor systems for monitoring the corrosion of the reinforcement in concrete structures. This sensor system indicates that the depth of the critical chloride content initiating the corrosion and thus the time to corrosion could be determined, enabling the owners of buildings to initiate preventive protection measures before cracks occur. Sicard et al (2003) studied corrosion monitoring of airframe structures using ultrasonic arrays and guided waves. A lamb wave phase velocity variation technique is developed to evaluate material thinning caused by corrosion.

2.2.3 Evaluation methods of corrosion damage

With the understanding on corrosion phenomenon and advance of corrosion research, the detection and monitoring techniques for stress corrosion damages have been developed and applied into practice. Moreover, the corresponding evaluation methods and monitoring strategies for stress corrosion damages are also developed to estimate and evaluate the potential damage caused by corrosion events. For civil engineering structures, the corrosion detection and evaluation of concrete structures and marine structures are the focus of attention due to their frequently reported corrosion damage in corrosive and humid environment.

corrosion prevention and remediation strategies for reinforced concrete coastal bridges in Oregon, USA. Razak and Choi (2001) carried out an experimental investigation to study the effect of general stress corrosion on the modal parameters of reinforced concrete beams. This investigation provides an insight into the use of modal parameters to detect damage in structural concrete elements which can be useful for structural appraisal and assessment purposes when applied to full scale structures. Trevor et al (2002) developed a statistical model to determine the time to first repair and subsequent rehabilitation of concrete bridge decks exposed to chloride deicer salts that incorporates the statistical nature of factors affecting the corrosion process. John et al (2002) introduced the use of permanent corrosion monitoring in new and existing reinforced concrete structures. Corrosion monitoring systems consists of linear polarisation, concrete resistivity and other probes to monitor durability in existing structures to evaluate rehabilitation strategies such as corrosion inhibitor application and patch repairs. The types of sensors used, data collection techniques, results and interpretation are provided. Eung et al (2003) carried out corrosion condition evaluation at muddy creek bridge. Montemor et al (2003) presented a state-of-the-art overview on the most important aspects of the corrosion process initiated by chlorides, its development and monitoring techniques. Robert (2003) studied the probabilistic model for marine corrosion of steel for structural reliability assessment.

In addition to the corrosion researches on concrete structures and marine structure, some works are also carried out to evaluate the corrosion damage of steel structures. Sarveswarana et al (1998) proposed a method to compute the component reliability of corrosion-damaged steel members using interval probability theory and the
method is illustrated using the data obtained from actual samples of corroded beams. McCuen and Albrecht (2004) conducted the re-analysis of thickness loss data for weathering steel in steel structure. Current researches on corrosion damage of steel structures, however, mainly focus on the structural components rather than the whole structural system. Very limited works have been carried out to evaluate the structural performance under corrosive environment. Fan and Xue (1994) conducted a field measurement on a steel space roof in China to examine the risk of SCC and assess structural load bearing capacity. The strong concern on the safety and function of some important steel space structures after a long-term service arises with establishment of some tremendous steel space structure across China in recent years for the coming Olympic Games. The assessment of structural safety and performance of steel space structure with corrosion damage is one practical problem. However, the evaluation and monitoring methods of corrosion damage of the entire steel space structure have not been systematically studied for the examination on structural performance and safety in harsh corrosive environment.

2.3 VIBRATION CONTROL

2.3.1 Development of structural control system

The structural control technique was initiated more than 300 years ago. In China, Palace in Beijing was base isolated with a layer of rice and many people believe this is one of the important reasons that it survived the mega earthquake in 19th century. It is during the Second World War that concepts such as vibration absorption and vibration damping were developed and effectively applied to aircraft structures. The structural engineering community seriously embraced this technology in the 1960s,
and since then many different paths have been developed. With the aid of developing theory in vibration, control and applied mechanics, many important modern structures around the world have been designed, or retrofitted with effective structural control systems. The new concepts of structural control, including passive control, active control, hybrid control and semi-active control have been growing. The application of these concepts in building design is accepted and it may preclude the necessity of allowing for inelastic deformation in the structural system (Housner et al., 1997). In the following, these four major types of control system are briefly reviewed.

2.3.1.1 Passive control system

Passive control systems are proved to be one of the effective methods in reducing the amplitudes of vibration by increasing the energy dissipation capacity of a structure through localized, discrete energy dissipation devices located within the structures. The passive control has many advantages: (1) it is usually relatively inexpensive; (2) it consumes no external energy; and (3) it is inherently stable. In general, such systems may be referred to as supplemental energy dissipation systems, which can be further characterized into three categories in accordance to the way of energy dissipation achieved: isolation system, vibration absorbing system and vibration damping system.

Seismic isolation systems represent one form of passive control systems. In these systems, a flexible isolation system is introduced between the foundation and superstructure so as to partially reflect and absorb some of the earthquake input energy before this energy can be transmitted to the structure. Isolation systems
generally consist of elastomeric bearings or sliding bearings, which are stiff and strong enough to support the vertical loads but quite flexible in the horizontal direction (Pong et al., 1994). Besides flexibility, these systems also have important energy absorbing capabilities provided by the high damping of the elastomeric bearings or the frictional behavior of the sliding bearings. This added damping becomes important to reduce the response in the case of excitations with significant energy in lower frequency ranges (Constantinou and Symans 1992). While large amplitude base drift normally limits its application in high-rise buildings.

The principle of vibration absorbing systems in energy dissipation is the transfer of energy among vibrating modes. The typical examples include tuned mass damper (TMD), tuned liquid damper (TLD) and tuned liquid column damper (TLCD). TMD basically consists of a moving auxiliary mass attached to the primary structure. The seismic input energy is transformed into kinetic energy of the moving mass, which is properly designed to induce dynamic forces opposing the motion of the primary system. In civil engineering applications, tuned mass dampers are usually tuned to the first natural period of the structural system. Therefore, they are most effective in those situations where the first mode contribution to the response is dominant (Soong, 1990). This is generally the case for tall and slender structural systems. Similar in the concept to a TMD, the TLD and TLCD impart indirect damping to the system and thus improve structural performance (Kareem, 1994).

The vibration damping systems utilize the motion of the structure to produce relative motion within damping devices to dissipate energy through mechanisms such as yielding of mild steel, viscoelastic action in rubber-like materials, sloshing of fluid,
shear of viscous fluid, orificing of fluid, and sliding friction. Typical vibration damping systems include metallic dampers, friction dampers, viscoelastic dampers and viscous dampers. The earliest implementations of metallic dampers in structural systems occur in New Zealand. A number of these interesting applications were reported by Skinner et al. (1980), and the typical examples of metallic dampers are X-shaped plate damper or added damping and stiffness device which were examined by Whittaker et al. (1991). As another mechanism for energy dissipation, the friction damper is firstly developed in the early of 1980 (Pall et al., 1980). The first installation of viscoelastic dampers is in the twin towers of the World Trade Center in New York to reduce wind induced acceleration responses. The viscoelastic dampers are evenly distributed throughout the structure from the 10th to the 110th floor. Significant efforts have also been directed towards the development of viscous fluid dampers for seismic mitigation. Examples of more recent structural applications of viscous fluid dampers can be found in Soong and Dargush (1997). A comprehensive introduction on passive control systems could be found in Soong and Constantinou (1994).

2.3.1.2 Active control system

An active control system is one in which an external source powers control actuator that applies forces to the structure in a prescribed manner. These forces can be used to both add and dissipate energy in the structure. In an active feedback control system, the signals sent to the control actuators are a function of the response of the system measured with sensors. In comparison with passive control systems, active control systems have a relatively short history. However, active control systems offer many advantages over passive control systems. These advantages include: (1)
enhanced effectiveness in motion control; (2) insensitivity to site conditions and ground motions; (3) applicability to multi-hazard mitigation situations such as strong wind and earthquake events; and (4) selectivity of control objectives such as structural safety or human comfort, for which a control system is designed. The versatility and adaptability of active control systems are the prominent features over the passive control systems. Therefore, the basic task is to find a control strategy associated with different analytical theories of control that uses the measured information to calculate the control signal that is appropriate to send to the actuators. Several analytical theories of active structural control have been developed in recent years such as optimal control, stochastic control, adaptive control, intelligent control and sliding mode control.

Optimal control is the design that involves minimizing or maximizing a performance measures under the dynamic system constraints (Skelton, 1988). The application of stochastic control principles to civil engineering structures is given with a broad acceptance because the stochastic control encompasses the function of enhancing a control policy for a dynamic system subjected to random disturbance under uncertainty of modeling (Soong, 1990). Adaptive control is generally used to control plants whose parameters are unknown or uncertain owing to the incorporation of a mechanism for adjusting the parameters of controller (Ioannou and Sun, 1996). Intelligent controllers, such as artificial neural networks control and fuzzy logic control, can be thought of as adaptive or self-organizing systems that learn through interaction with their environment (Widrow and Lehr, 1990; Zadeh, 1965). As a switching control method, sliding mode control was originally developed for the robust control of uncertain nonlinear system (Utkin, 1977, 1992). In addition, many
buildings and bridges have been implemented with full-scale active control system for structural protection against wind and earthquake (Spencer and Sain, 1997; Soong and Spencer, 2002). Considerable attention has been paid to the active structural control research in recent years, and actual systems have been designed, fabricated and installed in full-scale structures. Comprehensive reviews of active control systems can be found in Soong (1990), and Soong and Costantinou (1994). However, despite the above-mentioned advantages, active control systems require complicated signal processing systems and large force-generating equipment. The complexity reduces the robustness of the active control systems, and the large force requirement demands for a significant power resource. Such a power system may not be available during an earthquake event. Furthermore, the high cost of active control systems often limits their application.

2.3.1.3 Hybrid control system

Hybrid control system is basically a combination of passive and active systems. The resulting system can alleviate some of the limitations and restrictions associated with individual system and thus improve the overall performance of the controlled structure. A hybrid control system is often more reliable than a fully active system and yet adaptable to external excitations. One benefit of the hybrid control system is that, in the case of a power failure, the passive component of the control system still maintains a certain level of protection to the structure. A comprehensive review on hybrid control can be found in Housner et al (1997). To date, there exist many cases for full-scale implementations of hybrid control strategies. There are two main approaches for the implementation of hybrid systems: the hybrid mass damper (HMD) and the active base isolation. A hybrid mass damper combines a tuned mass
damper with an active actuator to enhance its robustness to reduce structural vibrations under different loading conditions. Usually the energy required by an HMD is far less than that required by an active mass damper with comparable performance. The V-shaped HMD is a typical HMD system developed by the Ishikawajima-Harima Heavy Industries (Koike et al., 1994), combining a pendulum passive mass damper installed on the 52-storey Shinjuku Park Tower. This system is also installed in both the Ando Nishikicho Building, a 14-storey building in Tokyo, and the Dowa Kansai Phoenix Tower, a 28-storey building in Osaka. Active base isolation, through the installation of additional active devices into seismic isolation systems, can achieve the isolation effect while keeping the base displacement at low levels. A great many of research studies have been performed to verify the effectiveness of this kind system in reducing structural responses (Kelly et al., 1987; Yoshida et al., 1994; Riley et al., 1998).

2.3.1.4 Semi-active control system

A semi-active control system is basically a passive control system whose mechanical properties could be adjusted based on the feedback from the excitation and/or from the measured responses according to a pre-determined control algorithm. In this sense, semi-active control devices are often considered as controllable passive devices. Typically, semi-active control devices require a considerably smaller power supply for operation than active control devices. Semi-active control systems are developed with the intent to have the best features of both passive and active control systems. They provide the reliability of passive devices, yet maintain the versatility and adaptability of active systems. The development and experimental testing of semi-active control systems for applications in structural response control has only
been pursued in recent years. Semi-active control systems are quite promising to be accepted as a viable means for protecting civil engineering structural systems against earthquake and wind loads. As indicated in the review paper by Housner et al (1997), appropriately implemented semi-active systems perform significantly better than passive systems and have the potential to achieve the performance of fully active systems, thus allowing for the possibility of effective response reduction during a wide range of dynamic loading conditions. Examples of semi-active control devices include variable stiffness devices, controllable friction devices, controllable tuned mass/liquid dampers and controllable fluid dampers.

Variable stiffness system was proposed by Kobori et al (1993), which avoids resonance of vibrating structural system by locking or unlocking the stiffness system. Its effectiveness has already been realized in a low-rise building. Several variable friction devices proposed by Akbay and Aktan (1990) and Kannan et al (1995) consist of a friction slider connected to the structural brace. Dowdell and Cherry (1996) also investigated analytically the ability of such semi-active friction devices in reducing the inter-storey drift of a seismically excited structure. Semi-active tuned mass damper is an integration of conventional TMD and a variable stiffness system which is developed to enhance adaptability of TMD. Controllable tuned liquid column dampers are developed to improve their effectiveness in vibration mitigation (Shum and Xu, 2005). Controllable fluid dampers utilize controllable fluids, which can reversibly change from a free-flowing, linear viscous fluid to a semi-solid with controllable static yield strength when exposed to electric or magnetic field. Electrorheological (ER) fluids and magnetorheological (MR) fluids are the most well-developed and viable fluids for development of controllable fluid dampers up to
now. The research and application of ER fluid dampers (Gavin et al., 1996a, b) and MR fluid dampers have been substantially carried out in recent years (Spencer et al., 1997; Jansen and Dyke, 2000; Johnson et al., 2003; Loh and Chang, 2006; Chang and Loh, 2006).

2.3.2 Passive and variable friction devices

Friction developed between two solid surfaces sliding against each other is prevalent in nature. It provides an effective and economical energy-dissipating mechanism by converting kinetic energy to heat. In reality, this concept has been successfully used by mechanical engineers to control the motion of machinery and automobiles. In the area of civil engineering, Pall et al (1980) pioneered the development of passive friction dampers for structural vibration suppression using this mechanism. Currently a wide variety of friction devices have been proposed and developed. Among them are the X-braced friction damper, the Sumitomo friction damper, the energy dissipating restraint and the slotted bolted connection. These devices vary significantly in terms of mechanical complexity and sliding materials. Pall et al (1980) developed the limited slip bolted joint and this device can be mounted on large panel structures for seismic mitigation. The X-braced friction damper proposed by Pall and Marsh (1982) is illustrated in Figure 2.3(a) and is used in buildings in Canada to provide the enhanced seismic protection of new and retrofitted structures. Displayed in Figures 2.3(b) and (c) are two uniaxial friction devices which are respectively proposed by Aiken and Kelly (1990) and Nims et al (1993). The configuration of the Sumitomo friction damper shown in Figure 2.3(b) reveals that it consists of copper alloy friction pads slide along the inner surface of the cylinder steel casing. The action of the spring against the inner and outer wedges can provide
the required clamping force. Energy dissipating restraint developed by Nims et al (Figure 2.3 (c)) consists of wedges, stoppers, and internal spring which can initiate the energy dissipation through sliding of the interface between bronze friction wedges and the steel cylinder wall. Another typical frictional device is slotted bolted connection (Figure 2.3(d)) which has been employed by Fitzgerald et al (1989) and Grigorian et al (1993) in the studies of vibration control of structural responses.

In general, passive friction dampers are simple to construct, yet provide satisfactory performance. They usually have rectangular energy dissipation loops, thus effectively increase the energy dissipation capacity of a structural system. In addition, passive friction dampers do not require replacement once they have served their purpose during an earthquake event (Pall and Pall, 1993). However, their metal interfaces typically promote additional corrosion. It is difficult to maintain the normal load on the sliding interface and some relaxation of the normal load is expected over the years. Compatible materials must be employed to maintain a consistent coefficient of friction over the lifetime of a damper (Housner et al., 1997). The passive friction damper should be designed with a specific optimum slip force under particular ground excitation, and therefore effectiveness of the friction dampers is seriously limited when real-time earthquake is far from the design target.

With the introduction of semi-active control, adjustment of the mechanical properties, clamping force, of the friction dampers can be achieved by low-level external energy source for instantaneous control of structure based on the response feedback, while the global stability of the structure can also be promoted simultaneously. The development of variable friction dampers for application in structural response
control in recent years has been based very much on semi-active approach. One variable friction damper proposed by Akbay and Aktan (1990, 1991) is composed of a shaft that is rigidly connected to the structure brace. The friction force is generated by allowing slippage occurred in the friction surfaces in a controlled manner. Kannan et al (1995) proposed a bang-bang control system to initiate early energy dissipation and thus maximize structural reduction. Dowdell and Cherry (1994, 1996) proposed two semi-active control algorithm, “Off-On” friction dampers and continuously variable “semi-active friction damper”, to realize vibration mitigation. Inaudi (1997) proposed a control strategy for semi-active friction devices called modulated homogeneous friction. This control algorithm is a collocated dynamic feedback law which uses the deformation of the damper as the only feedback signal to define the clamping force of the semi-active friction damper. Chen and Chen (2000) proposed another local-feedback control algorithm whose control logic is to provide a controllable contract force in accordance to both current states of displacement and velocity of the damper. Effectiveness of this algorithm is evaluated in a non-linear 20-storey benchmark building (Chen and Chen, 2002). Ng and Xu (2004) developed semi-active control to pursue increased vibration reduction of the building complex. Both global- and local-feedback control algorithms are considered in their numerical study.

Experimental investigation of variable friction damper for structural control also has been carried out for many years. Kannan et al (1995) designed a prototype active slip brace device and the clamping force is regulated by means of hydraulic mechanism. Hirai et al (1996) developed another variable friction damper system of which piezoelectric actuators were the primary element for modulation of clamping force
for such damper design. Shaking table tests were conducted by Pandya et al (1996) on a scaled four-storey structure. The variable friction dampers are placed on the diagonal brace systems of each storey and their normal forces are modulated by pneumatic actuators. Garrett et al (2001) developed a piezoelectric friction damper (Figure 2.4) and carried out characterization tests (Chen and Chen, 2004a). Similar to the design of variable friction damper, piezoelectric actuator is employed for clamping force regulation. Friction force is able to alter continuously by change of input voltage to the piezoelectric actuators. This piezoelectric friction damper has been experimentally examined by shaking table test for semi-active control of a quarter-scale three-storey building model with their proposed control algorithm (Chen and Chen, 2004b). Results demonstrate that piezoelectric actuator can provide an active and spontaneous variation of friction force at high control frequency. Different from the passive friction devices, the implementation of variable friction dampers in real buildings and structures is seldom reported due to its expensive cost and complicated operation which indicates more research and investigation is needed.

2.4 System Identification and Damage Detection

The interest in the ability to monitor a structure and identify structural parameters and damages at the earliest possible stage is pervasive throughout the civil, mechanical and aerospace engineering communities. System identification is defined as the process of developing or improving a mathematical model of a physical system using measurement data to describe the input, output and noise relationship. The first step towards this goal is the development of a mathematical model to represent the physical system. The second step consists of the estimation of the parameters involved in the mathematical model so that the mathematical model can
best reproduce the physical system. Various methods have been developed to improve the quality of the finite element model of a structure using measurement data. A detailed discussion of system identification methods can be found in a review paper by Mottershead and Firswell (1993). Structural damage can be defined as changes in structural parameters which adversely affect the current or future performance of the structure whereas structural damage detection aims to find such changes in the structure using measurement data. This section contains a review of the technical literature concerning the advances on system identification, damage detection and structural health monitoring in the past three decades. Doebling et al (1996) gave a comprehensive review on system identification and damage detection. Wu (2004) also discussed about the damage detection in detail in his thesis. The work of these two researchers formed part of my literature review in this section.

With regard to the algorithm used, these identification methods can be classified into index methods, model updating methods and signal based methods. The applications of the various methods to different types of engineering problems are categorized by types of structures and are summarised below. The types of structures include beams, trusses, plates, shells, bridges, offshore platforms, other large civil structures, aerospace structures, and composite structures. Regularization methods are utilized some times to improve the detection efficacy of identification results which is also illustrated.

2.4.1 Index methods

Index method is a group of detection approaches, which makes use of modal parameters as indicators to detect the structural damage location and extent. Some of these methods require the test modal data, such as modal frequency, mode shape, etc.,
to be correlated with the corresponding parameters of the finite element model (FEM), or to implement damage indicators with other information to localize the damage site. The methods are relatively simple and straightforward, but generally they do not provide quantitative information about the structural damage. The existing approaches may be classified into following categories based on modal parameters used in the damage detection: (1) index based on frequency changes; (2) index based on mode shape changes; (3) index based on mode shape curvatures/strain mode shapes; (4) index based on modal flexibility changes; (5) index based on modal strain energy changes; and (6) index based on frequency response function.

2.4.1.1 Index based on frequency changes

In the early stage of modal experiment, the technique and equipment for modal testing is not sophisticated and accurate enough, therefore, the most effective damage detection methods at that time are those using changes in natural frequencies. The reason is that natural frequency is the most fundamental vibration parameter, which can be obtained cheaply and reliably. The amount of literature related to damage detection using changes in natural frequencies is quite large. Vandiver (1975, 1977) examined the change in the frequencies associated with the first two bending modes and first torsional mode of an offshore light station tower to identify damage. Wojnarowski, et al (1977) examined the effects of eleven different parameters on the dynamic properties of an offshore lighthouse platform using finite element analysis. Foundation modeling assumptions, entrained water, marine growth, corrosion, variation in deck loads, and failed structural members are some of the parameters that are examined. Cawley and Adams (1979) presented a formulation to detect the
possible damage location in composite materials from frequency shifts. They started with the ratio between frequency shifts for modes \( i \) and \( j \), \( \frac{\delta \omega}{\delta \omega} \). In their method, the change in the natural frequency of a structure is a function of the damage position vector only, but not the damage extent. The simulated damage position, from which the analytical predicted ratio \( \frac{\delta \omega}{\delta \omega} \) equals the experimentally measured ratio, is indicated as possible damage site. The formulation does not account for possible multiple-damage locations. Penny et al (1993) proposed a method to locate the “most likely” damage case by simulating the frequency changes that would occur for all damage cases under consideration. The measured frequencies are then fit to the simulated frequencies for each simulated damage case in a least-squares sense. The “true” damage case is indicated by the minimal error in this fit. Slater and Shelley (1993) presented a method for using frequency-shift measurements to detect damage in a smart structure. They described the theory of modal filters used to track the frequency changes over time. Friswell et al (1994) presented a damage detection method based on a known catalogue of likely damage scenarios. They assumed that there exists a precise model of the structure, and compute the ratios of the frequency changes for several lower modes using the model of intact state and all the postulated damage scenarios. The same ratios are also calculated for the inspected structure. A power law relation is used to fit these two sets of values. When damage scenario of the structure lies in the set of assumed damages, the correct type of damage will produce a fit depicted by a unity-slope line. For all other types of damage, the fit will be inexact. Williams et al (1996) proposed another approach for improving damage localization using natural frequency sensitivity.
Zhang et al (1992) proposed another pattern recognition method for detecting structural defects in frame structures. The method is based on the fact that the ratios of relative change in natural frequency between any two modes are approximately equal to the ratio of the squares of the corresponding modal strain values at the damage position. Messina et al (1996) proposed a statistical-based assurance criterion from natural frequency change to detect single damage location through defining the damage location assurance criterion (DLAC) for location $j$:

$$DLAC(j) = \frac{|\Delta f^T \cdot \delta f_j^T|^2}{(\Delta f^T \cdot \Delta f) \cdot (\delta f_j^T \cdot \delta f_j)} \quad (2.1)$$

where $\Delta f$ is the measured frequency change vector for a structure with a single defect, and $\delta f_j$ is the theoretical frequency change vector for a damage of a known size at location $j$. The location $j$ with the highest $DLAC$ value demonstrates the best match to the measured frequency change pattern and is therefore considered as the damage site. They also extended this method to multiple damage detection and introduced algorithm for quantifying the extent of the damage (Messina et al., 1998) which is titled the multiple damage location assurance criterion (MDLAC).

Lifshitz and Rotem (1969) treated the damage detection as an inverse problem so that the damage parameters are calculated directly from measured frequency changes. Their works may be the first journal article to propose damage detection via vibration measurements. They found the change in the dynamic modulus, which is related to the frequency change, could be used as an indicator of damage in particle-filled elastomer. The dynamic modulus is computed by using curve fitting of the measured stress-strain relationships at various levels of filling. Stubbs and Osegueda (1990) developed a method for damage detection that relates changes in the resonant
frequencies to changes in member stiffness using a sensitivity relation. They assumed that damage occurs at only one member of the structure, and computed an error function for each mode and each structural member based on sensitivity analysis of modal frequency to damage. The authors illustrated that this sensitivity method has difficulty when the number of modes is much fewer than the number of damage parameters. Sophia and Garrett (1995) presented a technique for identifying localized reductions in the stiffness of a structure using measurements of natural frequency. The sensitivities of the eigenvalues to localized changes in the stiffness are developed as a set of underdetermined equations. The method is also verified using test data from an aluminum cantilever beam. Other examples of inverse methods for detecting damage via modal frequency changes were reported by Adams et al (1978), Wang and Zhang (1987), Hearn and Testa (1991), Koh et al (1995), Morassi and Rovere (1997) and Salawu (1997).

2.4.1.2 Index based on mode shape changes

Mode shapes inherently contain the spatial information about structural changes. Some researchers have devoted their efforts in locating damages with mode shape information. Most of the earlier methods proposed are based on the direct comparison of mode shapes, which are obtained before and after the structure is damaged. West (1984) presented what is possibly the first systematic use of mode shape information for the location of structural damage without the use of a prior FEM. The author used the modal assurance criteria (MAC) to determine the level of correlation between modes from the test of an undamaged space shuttle orbiter body flap and the modes from the test of the flap after it has been exposed to acoustic loading. The MAC, generally, is defined as
where \( \phi, \) and \( \phi_s \) are any two eigenvectors of a structural system. It stands for correlation level between the two modes on a scale from 0 to 1, with a value of 1 to indicate identical mode shapes and 0 for orthogonal ones. The mode shapes are partitioned using various schemes, and the change in MAC across the different partitioning techniques is used to localize the structural damage. Yuen (1985) used finite element analysis to obtain the natural frequencies and the mode shapes of the damaged structure in a numerical study. A systematic approach is used to determine the changes in mode shape due to the presence of structural damage. The study shows that a normalized set of eigenparameters describing the changes in the fundamental mode shape of the cantilever model possess definitive characteristics with respect to the location and extent of damage. Fox (1992) showed experimentally that measurement of mode shape changes for a single vibration mode, such as the MAC, is relatively insensitive to the saw cut damage in a beam. To locate the damage, the relative mode shape changes at node points are related to the corresponding peak amplitude points by a simple graphical comparison method. The author also presented a method of scaling the relative changes in mode shapes to better identify the location of the damage.

As a development to the MAC techniques, Lieven and Ewins (1988) proposed another correlation criteria using mode shape information for damage localization, named the coordinate modal assurance criterion (COMAC). Kim et al (1992) investigated the use of MAC and its variation in the location of structural damage. They used the partial MAC (PMAC) to compare the MAC values of coordinate
subsets of the modal vectors. By using the \textit{COMAC} and the \textit{PMAC} in conjunction, they were able to isolate the damaged area of the structure. Ko et al (1994) presented a method that uses a combination of \textit{MAC}, \textit{COMAC} and sensitivity analysis to detect damage in steel framed structures. The sensitivities of the analytically derived mode shapes to particular damage conditions are computed to determine which DOF are most relevant. The authors then analyzed the \textit{MAC} between the measured modes from the undamaged structure and the measured modes from the damaged structure to select which mode pairs to use in the analysis. Using the modes and DOF selected with the above criteria, the \textit{COMAC} is computed and used as an indicator of damage.

Other studies that examine the mode shape changes to identify damage are also reported. Mayes (1992) presented a method for model error localization based on mode shape changes known as structural translational and rotational error checking (STRECH). By taking ratios of relative modal displacement, STRECH assess the accuracy of the structural stiffness between two different structural degrees of freedom (DOF). Salawu and Williams (1994) compared the results of using mode shape relative changes and mode shape curvature changes to detect damage. They demonstrated that the relative difference measure does not typically give a good indication of damage using experimental data. Lam et al (1995) defined a mode shape normalized by the change in natural frequency of another mode as a “damage signature”. The damage signature is a function of crack location but not of crack length. They analytically computed a set of possible signatures by considering all possible damage states. The measured signatures are matched to a damage state by selecting which of the analytical signatures gave the best match to the measurements using the \textit{MAC}. Skjaeraek et al (1996) presented a structural damage detection
method based on changes in the natural frequencies and mode shapes using a substructure iteration technique. The optimal sensor placement issue for the method is also examined. Cobb and Liebst (1997) and Shi et al (2000) separately proposed a sensor location optimization method for their structural damage approaches based on eigenvector sensitivity analysis. Ratcliffe (1997) developed an approach, which uses a finite difference approximation of a Laplacian operator on mode shape measures, to localize the damage in a beam.

2.4.1.3 Index based on mode shape curvatures/strain mode shapes

Another class of damage identification method uses the mode shape derivatives such as curvatures to estimate changes in the dynamic characteristics of the structure. The mode shape curvature of a structure can be computed from the modal displacement or accelerations. Pandey et al (1991) demonstrated that absolute changes in mode shape curvature can be a good indicator of damage for the beam structures they consider. The curvature values are computed from the displacement mode shapes using the central difference approximation for mode \( i \) and DOF \( q \)

\[
\phi_{q,i}^* = \frac{\phi_{q-1,i} - 2\phi_{q,i} + \phi_{q+1,i}}{h^2}
\]  

(2.3)

where \( h \) is the length of each of the two elements between the DOF \((q-1)\) and \((q+1)\).

Chance et al (1994) found that numerically calculated curvature for mode shapes could introduce unacceptable errors. Instead of measuring or computing curvature directly, they used measured strains and achieved dramatic improvement in the results of damage identification. Nwosu et al (1995) evaluated strain changes with the introduction of a crack in a tubular T-joint. They found these changes to be much greater than any frequency shifts and to be measurable even at a relatively large
distance from the crack. Measuring the rotation of mode shapes is more difficult than measuring the translational mode shapes in the past. Recently, Abdo and Hori (2002) forecasted that the rotation of mode shapes might be feasible to be measured in the near future, as major advances have been made in the field of structural dynamics and mechanical vibration testing. They investigated the application of the rotation of mode shapes to detect and locate structural damage, and they found it is a sensitive indicator of damage. This demonstrates that the rotation of mode shapes has the ability to localize the damaged region when displacement modes fail.

2.4.1.4 Index based on modal flexibility changes

An alternative to using natural frequencies and mode shapes to obtain spatial information on changes in vibration is using their derivatives, such as modal flexibility. The measured flexibility matrix $\mathbf{G}$ is estimated from the mass-normalized measured mode shapes $\Phi$ and frequencies $\Lambda$ as

$$\mathbf{G} \approx \mathbf{\Phi} \Lambda^{-1} \mathbf{\Phi}^T$$  \hspace{1cm} (2.4)

A commonly accepted feature of modal flexibility is the fact that modal flexibility can be approximately estimated from a few of lower modes of the structure. As this feature inherently overcomes the shortcoming of mode incompleteness of measured modal data, many research efforts have been conducted on this subject.

Raghavendrachar and Aktan (1992) examined the application of modal flexibility for a three span concrete bridge. In their comparison, the modal flexibility is found to be more sensitive to local damages than natural frequencies or mode shapes. Aktan et al (1994) proposed the use of measured flexibility as a “condition index” to indicate the relative integrity of a bridge. They applied this technique to 2 bridges and compare
the measured flexibility to the static deflections induced by a set of truck-load tests. Pandey and Biswas (1994) presented a damage detection method based on changes in the measured flexibility of the structure. This method is applied to several numerical examples and to an actual spliced beam where the damage is linear in nature. Toksoy and Aktan (1994) computed the measured flexibility of a bridge and examined the cross-sectional deflection profiles with and without a baseline data set. Mayes (1995) used measured flexibility to locate damage from the results of a modal test on a bridge. He also proposed a method for using measured flexibility as the input for a damage-detection method (STRECH) which evaluates changes in the load-deflection behavior of a spring-mass model of the structure. Peterson et al (1995) proposed a method for decomposing the measured flexibility matrix into elemental stiffness parameters for an assumed structural connectivity. This decomposition is accomplished by projecting the flexibility matrix onto an assemblage of the element level static structural eigenvectors. Doebling et al (1996) proposed a technique to estimate unmeasured residual flexibility matrix. The residual flexibility matrix represents the difference between the exact flexibility matrix and the measured dynamic flexibility matrix, which is contributed from modes outside the measured bandwidth. Zhao and Dewolf (1999) presented a sensitivity study theoretically comparing the use of natural frequencies, mode shapes, and modal flexibilities for structural monitoring. Zhang and Aktan (1995 and 1998) suggested that changes in curvatures of the uniform load surface (ULS) calculated using the uniform load flexibilities, is a sensitive indicator of local damage. The authors stated that changes in the uniform load surface are appropriate to identify uniform deterioration. A uniform load flexibility matrix is constructed by summing the columns of the measured flexibility matrix. The ULS curvature is then estimated using a central
difference operator and used as an indicator to local damage. Wu and Law (2004) discussed the damage indices based on changes in modal flexibility, ULS and their curvatures and extended them to two dimensional space for localizing damage in plate structures.

2.4.1.5 Index based on modal strain energy changes

To achieve an effective approach to determine damage events, the mode shapes correlated with information of finite element model are utilized by some researchers to implement a new damage indicator. Some studies (Chen and Garba, 1988; Kashangaki et al., 1992) indicate that the strain energy is very useful in identifying structural damage. The general definition of modal strain energy of a structure with respect to the $i$th mode can be expressed as

$$MSE_i = \frac{1}{2} \phi_i^T K \phi_i$$  \hspace{1cm} (2.5)

where $\phi_i$ is the modal displacement shape of the $i$th mode, and $K$ is the stiffness matrix of a structure.

Stubbs et al (1992) presented the pioneer work on using modal strain energy to localize damage. They proposed a method based on examining the decrease in modal strain energy between two structural DOFs, as defined by the curvature of the measured mode shapes. Topole and Stubbs (1995) investigated the feasibility of using limited set of modal parameters for structural damage detection. Stubbs and Kim (1996) improved the method by using the modal strain energy to localize and estimate the severity of the damage without baseline modal parameters. Law et al (1998) presented another index named elemental energy quotient to detect damage events. Later, Shi et al (1998) proposed the concept of the elemental modal strain
energy (EMSE), they illustrated that with damage occurring in an element, the EMSE changes little in the intact elements, but there will be a larger change in the damaged elements. The modal strain energy change ratio can be a meaningful indicator for damage localization.

### 2.4.1.6 Index based on frequency response function

Another class of damage identification methods based on structural dynamic information uses the damage index from frequency response function (FRF) to estimate changes in the structural parameters. Samman et al (1991) used a scaled model of a typical highway bridge to investigate the change in FRF signals caused by the development of cracks in its girders. A pattern recognition method is introduced by utilizing the integer slope and curvature values of FRF wave forms, rather than peak magnitude. Only one FRF reading per girder is required to detect relatively minor cracks and locate crack approximately. Wang and Liou (1991) presented a new method to identify joint parameters by using the two sets of measured FRFs of a substructure with and without the effect of joints. Some strategies are applied to overcome the measurement noise problem that may result in false identification. Numerical simulation and experiments verify the accuracy of the proposed technique. Law et al (1992) developed sensitivity from a formulation based on the change in the FRF at any point, rather than just at the resonances. In practice, many points of the FRF around the resonances are taken, and a least squares fit is used to determine the changes in the physical parameters. Wu et al (1992) identified the damage in a three-storey building model by selecting the first 200 points of the frequency response function as input to a back propagation (BP) neural network. Biswas et al (1994) developed the modified chain-code method for the rapid detection of a small fault in
a structure. Slope and curvature based signatures are derived from the averaged composite FRF signature. The comparison of the intact signatures versus those of the cracked signatures can detect crack as small as 4 mm in hammer vibration tests of a bridge model. Chaudhry and Ganino (1994) used measured FRF data over a specified frequency range as input to a BP neural network to identify the presence and severity of delamination in debonded beams. Juan and Dyke (2000) presented and experimentally verified a new technique to identify damage based on changes in the component transfer functions of the structure, or the transfer functions between the floors of a structure. Multiple locations of damage can be identified and quantified using this approach.

2.4.2 Model updating methods

The algorithms solving for the updated matrices based on the governing equations of structural motion, the original model and the measured data are called model updating algorithms. Comparisons of the updated matrices to the original model correlated to the intact structure provide an indication of the location and extent of damage. In reality, the model updating algorithms are usually applied in both damage detection applications and model refinement applications in a similar way, namely, to seek an analytical model that is as close to the real structure as possible. However, considering the difference in application objectives between model updating and damage detection, attention should be paid to some particular issues for discriminating and relating the usage of model updating methods in the two fields mentioned above. The purpose of model updating is to modify the system stiffness, mass matrices and damping parameters of the numerical model so that better agreement between the numerical results and measured data can be obtained. The
damage detection applications aim to identify changes in stiffness, mass and damping matrices due to damage excluding the modeling errors. The major differences between the model updating and damage detection can be briefly explained as: the objective of the model updating is to obtain an improved finite element model so that model predictions match the measured data in an optimal way, while the damage detection is the process of locating areas of local stiffness or strength degradation in this context. The finite element model updating methods could be generally classified into the following three categories: (1) optimal matrix updating methods; (2) sensitivity based updating methods and (3) eigenstructure assignment methods. These model updating methods will be briefly reviewed in the following section.

2.4.2.1 Optimal matrix updating methods

Methods that use a closed-form, direct solution to compute the damaged model matrices or the perturbation matrices are commonly referred to as optimal matrix updating methods. Several reviews of these methods were published by Smith and Beattie (1991), Zimmerman and Smith (1992). The problem is generally formulated as a Lagrange multiplier or penalty-based optimization, which can be written as

$$\min_{\Delta M, \Delta C, \Delta K} \{ J(\Delta M, \Delta C, \Delta K) + \lambda R(\Delta M, \Delta C, \Delta K) \} \quad (2.6)$$

where $J$ is the objective function, $R$ is the constraint function, and $\lambda$ is the Lagrange multiplier or penalty constant.

Earlier work on optimal matrix updating using measured modal data was proposed by Rodden (1967), who used the measured vibration modes to determine the structural influential coefficients of an effectively unconstrained structure. Hall
(1970) presented an approach to optimize the stiffness matrix by minimizing the least-squares formed difference between the analytical modes and experimental modes based on an assumption that the mass matrix is exact. Constrained minimization theory has also been applied to the optimal matrix updating algorithms. Brock (1968) proposed an approach to optimize linear structural matrices by minimizing modal force errors with a property matrix symmetry constraint, which helps to preserve the reciprocity condition in the updated model. Baruch (1978), Kabe (1985), and Berman and Nagy (1983) had a common formulation of the optimal updating problem that is essentially minimization of the Frobenius norm of global parameter matrix perturbations using zero modal force error and property matrix symmetry as constraints. Kabe (1985) introduced a method that uses structural connectivity information as constraint to optimally adjust the stiffness matrix. Smith and Beattie (1991) extended the formulation of Kabe to include a sparsity preservation constraint and also formulated the problem as the minimization of both the perturbation matrix norm and the modal force error norm subject to the symmetry and sparsity constraints. McGowan et al (1990) also used structural connectivity information in their stiffness adjustment algorithms applied to damage identification, in which mode shape expansion algorithms are employed to extrapolate the incomplete measured mode shapes to be comparable with analytical predicted modes. Smith (1992) presented an iterative technique to the optimal update problem that enforces the sparsity of the matrix at each iteration step. The sparsity is enforced by multiplying each entry in the stiffness update by either one or zero, depending on the correct sparsity pattern.
Another approach to the optimal matrix updating is to minimize the rank of the perturbation matrix. Zimmerman and Kaouk (1994) revealed that perturbation matrices tend to be of small rank because damage is usually located in a few structural members rather than distributed all over the structure. They presented an algorithm based on the basic minimum rank perturbation theory that a unique minimum rank matrix solution for an underdetermined structure exists. Further research was extensively conducted by them and their colleagues to improve the algorithm (Kaouk and Zimmerman 1994, 1995; Zimmerman et al., 1995). Doebling (1996) presented a method to compute a minimum-rank update for the elemental parameter vector, rather than for global or elemental stiffness matrices. Mottershead and Shao (1991) utilized a cost function minimizing output errors to tune an analytical model. The output errors include frequency changes and response displacement changes. Gauss-Newton iterative approach is applied to solve the least squares problem. The applications of the optimal matrix updating method in model updating or structural damage detection have a long list in recent years. For example, Morassi and Rovere (1997) used only the frequency data before and after damage to detect a notch in a five-storey steel frame. Brownjohn and Xia (2000) tuned some parameters of substructures of a curved cable bridge to obtain an improved model.

2.4.2.2 Sensitivity-based updating methods

Another class of matrix update methods is based on the solution of a first-order Taylor series that minimizes an error function of the matrix perturbations. Such techniques are known as sensitivity-based updating methods. The basic theory is the determination of a modified parameter vector

\[ p^{k+1} = p^k + \delta p^{k+1} \]  

(2.7)
where the parameter perturbation vector $\delta p_{k+1}$ is computed from the iteration process for minimizing a prescribed error function.

An exhaustive classification of various sensitivity-based updating techniques was given by Hemez (1993). Jahn (1948) derived the complete formulae for eigenvalue and eigenvector sensitivities in a first-order Taylor series for a standard eigen-problem. The theory was then extended by Fox and Kappor (1968) in structural dynamics to solve the eigen-derivatives of a generalized symmetric eigen-problem with respect to physical variable changes. Some researchers try to improve the accuracy of the obtained approximate eigen-sensitivities in case where measurements are incomplete in modal orders. To essentially avoid such difficulties, Nelson (1976) developed an effective algorithm to compute eigen-derivatives of single mode by just using the modal data of that mode. Lim et al. (1987) presented an approximate modal method and Ting (1992) suggested an accelerated subspace iteration method to improve computational efficiency.

The earliest application of eigen-sensitivity analysis to finite element model updating was proposed by Collins et al. (1974). They generally formulated the inverse problem as a linear approximation below by using the truncated Taylor series of the modal data

$$\delta z = S \delta p$$

(2.8)

where $\delta p$ is the incremental vector to the updated physical parameters, $\delta z$ is the residual vector of the measured mode data, and $S$ is the eigen-sensitivity matrix. Chen and Garba (1980) then modified the method proposed by Collins by introducing matrix perturbation technique to avoid the eigen-solution required for
each iteration. Lin et al (1995) suggested employing both analytical and experimental modal data to calculate the eigen-sensitivities. Such accurately determined eigen-sensitivity coefficients are then used in the classical model updating procedure to overcome the existing difficulties of identifying small magnitude model errors and slow convergence. Law et al (2001) applied the super-element modeling technique to improve the finite element model of a bridge deck structure based on a similar consideration. The large number of DOFs in the original analytical model is dramatically reduced and the solution condition is improved.

The major difference between the various sensitivity-based methods is the modal parameters used to estimate the sensitivity matrix. In addition to the most popular natural frequencies and mode shapes, other types of data, e.g., FRF, time histories of responses, or combination of these, can also be used. Sanayei and Onipede (1991) proposed a parameter identification method in which the error function is the applied static force minus the structural stiffness matrix multiplied by the measured displacement. They proceeded on the numerical analysis of a truss and a frame structures. Sanayei and Saletnik (1996a) used strain error function, which is the difference between the analytical strains and measured ones, instead of static force error function. In the companion paper, Sanayei and Saletnik (1996b) applied this method and experimented on a truss and a frame to perform parameter estimation. Ziaei-Rad (1997) developed a FRF-based model updating by expanding the inverse matrix of FRF as a Taylor series function with respect to structural parameters. Numerical examples and experimental examples are applied to update the analytical model so that the FRFs match those obtained in testing. Ziaei-Rad and Imregun (1996) discussed the experimental error bounds for convergence of this updating
algorithm. Fritzen et al (1998) developed another model updating based damage
detection method using the sensitivity of FRFs with respect to damage. Abdel (2001)
presented a damage detection method based on model updating, in which the
sensitivity of the natural frequencies, mode shapes and modal curvatures to damage
are combined to construct the sensitivity matrix. A structural damage detection
method through the sensitivity-based finite element model updating procedure was
presented by Hemez and Farhat (1995). They combined modal force error and static
force error as error function to identify structural damage.

2.4.2.3 Eigenstructure assignment methods

Another matrix updating method, known as “eigenstructure assignment”, is based on
the design of a fictitious controller which would minimize the modal force error. The
controller gains are then interpreted as parameter matrix perturbations to the
undamaged structural model. In eigenstructure assignment approaches, the model
updating problem is formulated as a closed loop system, in which the state feedback
describes the right hand side of the equation of motion in terms of the displacement
and velocity variables. The feedback gain matrix is determined so that the output
eigenvalues and eigenvectors correlate the measured eigen-data very well. This
procedure will result in modifications to the stiffness and damping matrices but the
mass matrix remains unchanged. The equation of motion with a controller is
represented by

\[ M\ddot{\mathbf{x}} + C\dot{\mathbf{x}} + K\mathbf{x} = \mathbf{F}\mathbf{u} \]  

(2.9)

where \( \mathbf{u} \) is the control force vector and \( \mathbf{F} \) is a control influence matrix. Suppose that
the control gains are selected such that the modal force error between the nominal
structural model and the measured modal parameters from the damaged structure is zero. Defining

$$L_{ij} = ((\lambda_i^d)^2 M + \lambda_i^d C + K)^{-1} F^T_j F_j$$

(2.10)

Then the “best achievable eigenvectors” can be written in terms of the measured eigenvectors

$$\varphi_{ij}^* = L_{ij}^* \varphi_i$$

(2.11)

where $+$ means pseudo-inverse. The relationship between the best achievable eigenvectors and the measured eigenvectors is then used as a measure of damage location.

Minas and Inman (1990) proposed two model updating methods based on eigenstructure assignment technique. The first method formulates the problem as a non-linear optimization procedure with enforced symmetry constraint. The second method uses only the eigenvalue information incorporating with a state-space formulation to determine the state matrix. Zimmerman and Widengren (1990) also presented a symmetric eigenstructure assignment approach in which a generalized algebraic Riccati equation is used to calculate the stiffness and damping correction matrices. Zimmerman and Kaouk (1992) firstly adopted eigenstructure assignment technique to conduct damage detection. They used a subspace rotation algorithm to improve the assignability of the eigenvectors and preserve matrix sparsity in the updated model. Lindner and Goff (1993) used an eigenstructure assignment technique to identify the damage coefficient defined for each structural member. A numerical simulation is performed to detect damage in the finite element model of a ten-bay truss structure. Lim (1994, 1995) applied a constrained eigenstructure assignment technique to process the measured incomplete modal data from a twenty-
bay planar truss. The feedback gain matrix is diagonalized, and the diagonal members are interpreted as element-level perturbations to the stiffness matrix so that the damage is localized directly. Lim and Kashangaki (1994) deduced the best achievable eigenvectors considering reduction of both stiffness and mass properties. They applied this technique to detect the damage in an eight-bay cantilevered truss by selecting elements with higher strain energy respect to a mode. Schulz et al (1996) presented a technique similar to eigenstructure assignment known as FRF assignment. The authors formulated the problem as a linear solution for element-level stiffness and mass perturbation factors. They illustrated that using FRF measurements directly to solve the problem is more straightforward than extracting mode shapes. The eigenstructure assignment based method was also adopted by Cobb and Liebst (1997) in structural damage detection.

2.4.2.4 Stochastic model updating methods

One important issue in model updating methods is the statistical variation both in measured modal properties and in material properties. The uncertainties in the measured modal properties may result from noise contamination or the uncertainties in structural parameters. The measured modal properties are inevitably corrupted with noise contamination no matter how precise the instrumentation is. Furthermore, the errors can also be introduced when identifying the modal properties from the time-domain signals such as accelerations. In addition, many structural parameters such as Young’s modulus and material strength are inherently random variables that may be statistically investigated. In the presence of uncertain modal properties, it is important to study and quantify their effects on the model updating results, as well as to estimate the resulting statistics of updating parameters. The stochastic model
updating or statistical system identification can be described as the statistical determination of structural parameters in the updated FE model by an uncertainty-propagation approach based on the statistics of measured modal data.

The commonly developed approach for uncertainty propagation is the Monte Carlo Simulation (MCS) method which first generates large amounts of samples following the predefined probability density functions (PDFs) or the joint PDFs of modal properties. Then, the model updating algorithm is repeatedly carried out for these samples to obtain the corresponding solution samples of updating parameters. The PDFs of updating parameters are finally determined based on the solution samples. Agbabian et al (1988) employed the MCS method to identify the statistical properties of stiffness coefficients in a linear system. They generated the time histories of the applied excitation as well as the accelerations, velocities, and displacements of a system. By separately applying the model updating procedure to different time segments, ensembles of stiffness coefficients were identified. Their work has been later extended to statistical identification of a nonlinear system approximated by an equivalent linear system (Smyth et al., 2000). Banan et al (1994a, b), Sanayei and Saletnik (1996a, b), Yeo et al (2000) and Zhou et al (2003) adopted similar approaches for studying the effects of measurement noise on identification results.

However, the MCS method is by itself computationally intensive as it requires a large number of simulations to obtain accurate and valid statistics. In this regard, another technique for uncertainty propagation, perturbation method, is developed and applied in stochastic structural analysis where the perturbation technique in conjunction with the FE analysis is applied to evaluate the response variability and
failure probabilities associated with prescribed limit states (Kleiber and Hien 1992). Perturbation method expands the nonlinear function in terms of random variables either by a linear function or by a quadratic one at a particular point. Second moment technique is then applied to evaluate the mean and standard deviation of the response, or to evaluate the failure probabilities. Liu (1995) described the identification of structural parameters as a linear least squares problem in order to minimize the modal force residuals. Each term is expanded in a system of linear equations in terms of random modal properties. Making use of the expanded sets of linear equations, the mean and covariance of updating parameters are finally derived. Papadopoulos and Garcia (1998) presented a two-step probabilistic method for damage assessment in order to determine the statistics of stiffness coefficients of the damaged structure. Xia et al (2002), and Xia and Hao (2003) updated the statistics of stiffness of the damaged structure in a single step and used the statistics of stiffness before and after structural damage to implement probabilistic damage detection. Other researches addressing the statistical parameter estimation using uncertain modal data include the work of Katafygiotis and Beck (1998), Li and Roberts (1999a, b), Araki and Hjelmstad (2001), Fonseca et al. (2005) and Yuen and Katafygiotis (2005).

2.4.3 Signal based methods

The widely used vibration based damage assessment methods require modal properties that are obtained from signals via the traditional Fourier transform (FT). There are a few inherent characteristics of FT that might affect the accuracy of damage identification. Firstly, the FT is in fact a data reduction process and information about the structural health condition might be lost during the process (Doebling et al., 1998). Also, the FT is not able to present the time dependency of
signals and it cannot capture the evolutionary characteristics that commonly observed in the signals measured from naturally-excited structures (Gurley and Kareem 1999). Doebling et al (1998) also stated that damage is typically a local phenomenon which tends to be captured by higher frequency modes. These higher natural frequencies normally are closely spaced but poorly excited. All these factors add difficulties to the implementation aspect of the FT based damage detection techniques. To overcome this shortcoming, many signal based identification methods such as wavelet transform (WT), Hilbert-Huang transform, auto-regressive moving-average (ARMA) family models have been developed in recent years which will be briefly reviewed in the following section.

2.4.3.1 Wavelet transform

WT can be viewed as an extension of the traditional FT with adjustable window location and size. It allows an arbitrary function to be expressed as a series expansion where each term is one of the basis wavelets multiplied by its magnitude. Compared with the Fourier-based analyses that use global sine and cosine functions as bases, the basis wavelets are local functions, each of which is defined by two parameters: its scale (relating to frequency) and its position (relating to time). The use of local functions allows a simultaneous and varying time-frequency resolution that leads to a multi-resolution representation for non-stationary data (Gureiy and Kareem 1999). In summary, FT technique has been implemented in the research area of structural damage assessment for a very long period, but, its application is restrained by its own properties and some research issues cannot be solved profoundly. Hence, a newly developed tool, WT, which owns better properties, can be put in use to investigate
the possibility to solve existing problems and expand the application area of vibration-based damage assessment methods.

Morlet, a geophysicist, and Grossmann, a theoretical physicist initially proposed wavelet theory. Together with their fellow Frenchman, Meyer (1992), their ‘French school’ developed the mathematical foundations of wavelets. At this stage wavelets are still very much in the realms of pure mathematics and concentrated more on the theory than on the application. The two American researchers Daubechies (1992) and Mallat (1998) changed this by defining the connection between wavelets and digital signal processing. Wavelets have been applied to a number of areas, including data compression, image processing and time-frequency spectral estimation. The earliest work on applying wavelet analysis in structural damage detection dates back to the work of Yamamato and his group in 1995, in Kyoto, Japan. Joint research work by Yamamato's group and a group at Worcester Polytechnic Institute (WPI), headed by Noori and Hou, collaborated in advancing this new technique further in 1996. Wavelet analysis has been one of the fastest evolving signal processing tools in the area of damage detection. The applications of wavelet analysis in damage detection includes many aspects such as time-frequency analysis, wavelet spectrum, orthogonal or discrete wavelet decomposition, wavelet-based data compression, de-noising and feature extraction, linear and nonlinear system identification, image processing. Jacob, Desforges and Ball (1997) discussed the use of wavelet coefficients as features for damage identification. Because material damage in composites often produces high frequency acoustic emissions, the authors applied WT to the acoustic emission time signals to identify the failure mode of the material. Naldi and Venini (1997) briefly explored the use of wavelets to detect damage in
structural components. The authors numerically simulated damage in a beam constrained in all but the axial direction. The coefficients of the Daubechies wavelet is used to locate the damage, and a harmonic excitation applied to the beam. Wang and Deng (1999) discussed a structural damage detection technique based on wavelet analysis of spatially distributed structural response measurements. This approach is based on a premise that damage in a structure causes the structural response perturbations at the damage sites, and the local perturbations are often discernable in wavelet components. Effects of noise contamination and damage severity on damage detection were discussed (Ai-kbalidy et al., 1997a, b). Faults in gear systems are detected using wavelet approaches and some results are verified by an inspection (Wang and McFadden 1995; Staszewski and Tomlinson 1997; Ferlez and Lang 1998; Samuel et al., 1998). The wavelet approach for on-line detection of an abrupt stiffness loss is studied and the results were compared with other approaches such as a neural network based on-line approximation technique and the EMD method (Vincent et al., 1999; Demetriou and Hou 1999; Hou and Noori 1999, 2000; Hera and Hou 2002, 2004).

As an extension of WT, the wavelet packets transform (WPT) is particular linear combinations of wavelets which provides excellent resolution for the analysis of high frequency signal components and has been successfully applied in image classification, damage assessment and health monitoring. Wavelet packets use a rich library of redundant bases with arbitrary time-frequency resolution and hence own additional advantage of providing excellent resolution for the analysis of high frequency signal components. This property makes it fit for the requirements of damage detection because most of information related to structure damage is hidden.
in the high frequency range. Owing to above summarised advantages, WPT has been viewed as a powerful tool in signal and image classification (Learned and Willsky 1995), machining process monitoring (Wu and Du 1996), tool wear monitoring (Li and Yuan 1998), vibration monitoring (Yen and Lin 2000) and damage detection (Sun and Chang 2002, 2004).

2.4.3.2 Hilbert-Huang Transform

More recently, Huang (Huang, 1996, 1998, 1999) developed a new method named Hilbert-Huang transform (HHT) for analyzing nonlinear and non-stationary data. HHT method consists of EMD and Hilbert transform (HT). The most innovative idea is the introduction of EMD method with which any complicated set of data can be decomposed into a finite and often small number of intrinsic mode functions (IMF) that admit well-behaved HT. Since the EMD is based on the local characteristic time scale of the original data, this decomposition method is adaptive and highly efficient. The EMD method, which is once named as characteristic scale decomposition method (Huang, 1998), is initially designed as a preprocessing step to extract simple oscillatory functions from the signal. The EMD builds on the assumption that any data set consists of different, simple, intrinsic modes of oscillation. Then, any data set can be decomposed into several IMFs by a sifting process. The basis of this decomposition is based on and derived from the data, therefore, each IMF component is physically meaningful.

Having obtained the IMF components from the time history $x(t)$, the second step of the HHT method is implemented by performing the HT to each IMF component. HT has long been used for linear and nonlinear vibration system identification for many
years. The amplitude, instantaneous phase and varying frequency can be obtained by applying HT to vibration signals to identify the properties of a certain vibration system. The HT of a real-valued function \( y(t) \) in the range \(-\infty < t < \infty\) is a real-valued function \( \tilde{y}(t) \) defined by (Bendat and Piersol 2000)

\[
\tilde{y}(t) = H[y(t)] = \int_{-\infty}^{\infty} \frac{y(u)}{\pi(t-u)} du
\]  
(2.12)

Where \( H \) denotes the HT operator. Essentially, the HT is the convolution of \( y(t) \) with \( 1/t \), which emphasizes the local properties of the data signal. The corresponding analytic signal \( z(t) \) can then be defined as

\[
z(t) = y(t) + i\tilde{y}(t) = A(t)e^{i\theta(t)}
\]  
(2.13)

\[
A(t) = [y^2(t) + \tilde{y}^2(t)]^{1/2}
\]  
(2.14)

\[
\theta(t) = \tan^{-1}\left[\frac{\tilde{y}(t)}{y(t)}\right]
\]  
(2.15)

where \( A(t) \) and \( \theta(t) \) are defined as the amplitude and instantaneous phase angle of \( y(t) \), respectively; and \( i \) is the imaginary unit. The instantaneous frequency \( \omega(t) \) is then given by

\[
\omega(t) = \frac{d\theta(t)}{dt}
\]  
(2.16)

From Equations (2.12)-(2.16), there is only a single frequency \( \omega \) at any time \( t \) if the general signal \( x(t) \) is processed through the HT. For a general signal \( x(t) \) at any time \( t \), there is a distribution of frequencies at that moment rather than just a single frequency. Consequently, Huang et al (1998) pointed out that the definition of instantaneous frequency for a signal has physical significance only if it is an IMF. This is the reason why a signal is decomposed into the IMFs using the EMD method before applying the HT. After applying the HT to each IMF
component \( c_j (j = 1, 2, \ldots, N) \), the original time history excluding the final residue can be then expressed as the real part (Re) of the sum of the HT of all the IMF components

\[
X(t) = \text{Re} \sum_{j=1}^{N} a_j(t) e^{i \int \omega_j(t) dt}
\]  

(2.17)

where \( a_j(t) \) and \( \omega_j(t) \) are the amplitude and frequency of the \( j \)th IMF component respectively. Thus, the amplitude is not only the function of time but also the function of frequency. The frequency-time distribution of the amplitude is designated the Hilbert amplitude spectrum, \( H(\omega, t) \), or simply the Hilbert spectrum, from which the inherent characteristics of a nonlinear and/or nonstationary time history can be identified.

The HHT method has then been quickly used in many scientific and engineering disciplines, such as investigation on the characteristics of gravity waves in middle atmosphere, application to examine the time-dependent water surface elevation of water waves (Schlurmann, 2001) and to remove a variety of artifacts from gastric signal in electrogastrogram recordings. In all these studies, the HHT method gives new insights into the non-stationary and nonlinear physical phenomena. The HHT method has been used in structural engineering analysis as well, especially for structural damage detection and structural dynamic parameters identification. Application of EMD to structural damage detection was investigated by Vincent (1999) and compared with the WT approach. Sudden stiffness loss damage is utilized in the numerical studies. The results show that both the EMD and WT approaches can identify the time at which the damage occurs, while one advantage of the EMD method is its adaptive nature. Yang (Yang 1999, 2000, 2003a, 2003b) used EMD
method and HT to identify the structural frequency and model damping ratio. In his method, the measured response of MDOF structures is first decomposed into IMF using EMD. Then HT is applied to each IMF resulting in the frequency-time domain, from which the natural frequencies and damping ratios are directly determined. To address the effect of measurement noise, they developed the method by introducing the random decrement technique to each IMF to obtain the free vibration modal responses, and then applied the HT to obtain the frequency-time representative of the original signal. Finally, natural frequencies and damping ratios are estimated from the instantaneous phase angle and decay amplitude using linear least-square fit procedures. Numerical studies show that the methodology is a simple effective and accurate tool for parameter identification of linear structures. The possibility of using the EMD-HT method for modal parameter identification of linear system with closely spaced modes of vibration was also investigated by Chen and Xu (2002). The results showed that the EMD-HT method is more capable of identifying modal parameters when the natural frequencies are close to each other than either fast FT-based method or the WT method. Yang et al (2001, 2004) also proposed a HHT based approach for structural damage detection. Xu and Chen (2003, 2004) conducted a comprehensive experimental investigation on the applicability of EMD for identifying structural damage caused by a sudden change of structural stiffness. The experiments are carried out on a 3-storey shear building model installed on a seismic simulator with two springs horizontally connected to the first floor of the building to provide additional structural stiffness. Structural damage is simulated by suddenly releasing the two pre-tensioned springs either simultaneously or successively. Various damage severities are produced using springs of different stiffness. The identified results are compared with the results measured directly from
the experiments to verify the EMD approach. Chen and Xu (2004) also applied the EMD-HT method to the field measurement data of the Tsing Ma suspension bridge recorded during Typhoon Victor to identify the natural frequencies and modal damping ratios of the bridge and to perceive their changes with wind speed and bridge vibration amplitude. The identified results of EMD-HT are compared with those obtained by the fast FT-based method to evaluate the applicability of the EMD-HT method to both stationary and non-stationary bridge responses during typhoons. In addition, EMD combined with HT were also used by Loh et al (2001) to analyze the free-field ground motion and to estimate the global structural property of buildings and bridges through the measurement data of seismic response. By application of the EMD-HT method to seismic response data of buildings and bridges, the time-varying natural frequency and damping ratio of the system could be estimated. Damage identification from seismic response data of buildings and bridges, particularly from the Chi-Chi earthquake data, was discussed as well.

2.4.3.3 ARMA family models

An alternative approach of damage identification based on dynamic response signals are proposed ARMA family models. Heyns (1997) used a multivariate auto-regressive vector (ARV) model to detect and locate damage in a cantilever beam with two different widths. Heyns noted that this ARV approach does not require artificial excitation or measurement of the input forces. Mode shape changes are extracted from the AR model, and curvatures are calculated from the mode shape changes. The curvatures are determined to be more sensitive to damage than are mode shapes. De Stefano, Sabia, and Sabia (1997) used auto-regressive moving-average vector (ARMAV) models to obtain modal parameters of a three-span bridge.
girder with unknown random excitation. To avoid solving a complicated nonlinear optimization problem, the authors fit an ARV model to the data and compute the AR coefficients and prediction errors. The AR and MA terms are found by means of a least squares approach. This process is repeated a few times until convergence is reached to obtain the modal properties. Sohn and Farrar (2000) applied statistical process control techniques to vibration based damage diagnosis. First, an AR mode is fit to the measured acceleration time histories from an undamaged structure. Then, the AR coefficients obtained from subsequent new data are monitored relative to the baseline AR coefficients. Any significant deviation from the baseline of AR coefficients would indicate either a change in environmental conditions or damage. The statistical procedure combined with the AR time prediction model is applied to vibration test data acquired from a concrete bridge column as the column is progressively damaged. Bodeux and Golinval (2000) demonstrated the use of ARMAV models on the Steel-Quake structure at the Joint Research Center in Ispra, Italy. The Steel-Quake structure is used to test the performance of buildings during earthquakes. Bodeux and Golinval detected damage in the Steel-Quake structure using changes in the frequencies estimated by the ARMAV technique. Their statistical approach is based on the mean and sample standard deviation of the estimated natural frequencies. Loh and Wu (2000) presented an experimental investigation of the Fei-Tsui arch dam through forced vibration tests and the analysis of seismic response data. In the identification of dam properties from seismic response data, the multiple input/multiple output discrete-time ARX model with least squares estimation was applied to consider the nonuniform excitation of the seismic input and the global behavior of the dam. To verify the accuracy of the identification
results, comparisons between the discrete-time ARX model and a frequency domain conditioned spectral analysis were made.

2.4.4 Regularization techniques

2.4.4.1 Statement of the problem

A stable identification algorithm is essential for a reliable damage assessment. Unfortunately, the identification equations for many damage assessment methods are normally troubled with the ill-condition of the coefficient matrix characterized by nonuniqueness and discontinuity of solutions when the measured data are polluted with random error (Bui 1994; Hjelmstad 1996). The ill-condition of identification equation for damage assessment and modal updating has attracted more attention in recent years. Natke (1992a, 1992b) advocated the application of regularization techniques in model updating. Fregolent et al (1996) considered a variety of methods for determining the regularization parameter in the equation error problem. Friswell et al (1995) pointed out that the regularization has become a central issue in system identification, because the dynamic behavior is observed in a narrow knowledge space and, consequently, the systems of equations are strongly under-determined. Ahmadian et al (1998) revealed that many model updating problem are ill-conditioning which makes the identified solution very sensitive to the measurement error and further cause the fluctuation of obtained results. They investigated the problem of selecting a side constraint and determining the regularization parameter in model updating. The results demonstrate that the weight to be attached to the constraint is determined by the regularization parameter. It is found that the method of cross validation can be used reliably to truncate the small generalized singular values which contain the measurement noise. The L-curve approach is similarly
robust in locating the regularization parameter, and this is demonstrated in a physical experiment. It is shown that careful selection of the side constraint can lead to updated parameters with physical understanding. Yeo et al (2000) presented a damage assessment algorithm for framed structures based on a system identification scheme with a regularization technique. The regularization technique is introduced to alleviate the ill-conditioning of the system identification problems. A new regularization function based on the Frobenius norm of the difference between the estimated and the baseline stiffness matrix is proposed and the validity of the proposed method is further demonstrated through numerical examples.

The identification equation for damage assessment or model updating is normally expressed as

$$A\theta = b$$  \hspace{1cm} (2.18)

where $$A \in \mathbb{R}^{n \times m}$$, $$b \in \mathbb{R}^{m \times 1}$$, $$\theta \in \mathbb{R}^{n \times 1}$$ and the parameter $$\theta$$ are required. If consider the case when $$b$$ is contaminated with random error, $$\varepsilon$$, having zero mean and with mutually independent entries. The least-square solution $$\theta_{ls}$$ is unique and unbiased provided that $$\text{rank}(A) = m$$. When $$A$$ is close to being rank deficient then a small $$\varepsilon$$ may lead to a large deviation in $$\theta$$ from its exact value and the solution is said to be unstable and Equation (2.18) is ill-conditioning. A different problem occurs when $$m > n$$ so that Equation (2.18) is underdetermined and there are an infinite number of solutions. The pseudo-inverse solution of minimum norm can be obtained by using singular value decomposition (SVD). The primary difficulty with the ill-condition of identification equation is the underdetermined problem. It is therefore necessary to incorporate further information about the desired solution in order to stabilize the problem and further obtain a useful and stable solution which is actually the purpose
of regularization. Although many types of additional information about the solution \( \theta \) is possible in principle, the dominating approach to regularization process is to require that the 2-norm (or an appropriate seminorm) of the solution be small.

### 2.4.4.2 Regularization methods

The most common and well-known form of regularization is the Tikhonov regularization (1977) whose basic idea is to define the regularized solution \( \theta(\lambda) \) as the minimizer of the weighted combination of the residual norm and the side constraint (quadratic cost function)

\[
\| A\theta - b \|_2^2 + \lambda \| L \theta - d \|_2^2
\]

where \( L \in \mathbb{R}^{p \times m} (p \leq m) \) is chosen so that

\[
\text{rank} \begin{bmatrix} A \\ L \end{bmatrix} = m
\]

which is an expression of Morozov’s complementation condition (Morozov 1984) and \( \lambda > 0 \) is the regularization parameter (some authors use \( \lambda^2 \) in place of \( \lambda \)). The basic idea is to minimize the cost function in Equation (2.19) by searching for a solution \( \theta(\lambda) \) which at the same time produces a small residual \( \| A\theta - b \|_2^2 \) and a moderate value of the side constraint \( \| L \theta - d \|_2^2 \). The way in which these two terms are balanced depends on the size of the regularization parameter \( \lambda \). If \( \lambda \) is too small then the problem will be too close to the original ill-posed problem, but if \( \lambda \) is too large then the problem solved will have little connection with the original problem.

The matrix \( L \) is typically either the identity matrix \( I_m \) or a discrete approximation to a derivative operator (Phillips 1962). The correct choice of \( L \) is important to obtaining meaningful parameters \( \theta \). Varah (1983) showed that a wrong choice of \( L \)
can lead to completely erroneous results. The additional information should be introduced by means of the side constraint in the model updating or damage assessment process.

The other normally adopted the regularization techniques is the generalized singular value decomposition (GSVD). Considering the system of equations in the GSVD (Hansen 1994),

\[
\begin{bmatrix}
A \\
L
\end{bmatrix}
\begin{bmatrix}
\theta \\
\delta
\end{bmatrix} = \begin{bmatrix}
b \\
d
\end{bmatrix}
\] (2.21)

The decompositions of \(A\) and \(L\) are in the form

\[
A = U \begin{bmatrix} I & \Sigma \end{bmatrix} X^{-1}
\] (2.22)

\[
C = V [0 \quad M] X^{-1}
\] (2.23)

where \(X \in \mathbb{R}^{n \times m}\) is non-singular, and the columns of \(U \in \mathbb{R}^{n \times m}\), \(V \in \mathbb{R}^{p \times p}\) are orthogonal and \(n \geq m \geq p\). Matrices \(\Sigma\) and \(M\) are

\[
\Sigma = \text{diag}(\sigma_1, \sigma_2, \ldots, \sigma_p)
\] (2.24)

\[
1 \geq \sigma_1 \geq \sigma_2 \geq \ldots \geq \sigma_p \geq 0
\] (2.25)

\[
M = \text{diag}(\mu_1, \mu_2, \ldots, \mu_p)
\] (2.26)

\[
0 \leq \mu_1 \leq \mu_2 \leq \ldots \leq \mu_p \leq 1
\] (2.27)

And the terms \(\sigma_i, \mu_i (i = 1, 2, \ldots, p)\) are normalized so that

\[
\sigma_i^2 + \mu_i^2 = 1
\] (2.28)

The generalized singular values of \(\begin{bmatrix} A \\
L
\end{bmatrix}\) are then given by

\[
\gamma_i = \frac{\sigma_i}{\mu_i} \quad (i = 1, 2, \ldots, p)
\] (2.29)
in decreasing order. The columns of \( \mathbf{X} \) relating to the largest generalized singular values span the range of \( \mathbf{A} \) and the null space of \( \mathbf{L} \). The reverse is true of the smallest generalized singular values. Morozov’s complementation condition states that the range of \( \mathbf{L} \) should contain the null space of \( \mathbf{A} \). Therefore, the aim is to truncate the singular values at \( i = j \) so that \( \| \mathbf{A}\mathbf{0} - \mathbf{b} \|_2 \leq \sigma_j \) and \( \| \mathbf{L}\mathbf{0} - \mathbf{d} \|_2 \) is a minimum. This can be achieved by applying Picard’s condition (Hansan 1994) to truncate the singular values when \( (\mathbf{u}_i^\top \mathbf{b} / \sigma_i)_{i,j} \) takes a large value.

2.4.4.3 Determination of regularization parameters

(1) Cross validation

The idea of cross validation is to maximize the predictability of the model by choice of the regularization parameter \( \lambda \). A predictability test can be arranged by omitting one data point, \( b_k, k = 1, 2, \ldots, n \), at a time and determining an estimate \( \theta(\lambda) \), using the other data points. Then for each of the estimates, predict the missing data and find the value of \( \lambda \) that on average predicts the \( b_k, k = 1, 2, \ldots, n \), best. This is the method of cross validation (Stone 1974). The basic procedure is briefly described in the following steps (Friswell et al., 1995).

(a) Find the estimate \( \theta(\lambda) \) which minimizes

\[
\sum_{i=1, j \neq k}^{m} \left( b_i - \sum_{j=1}^{m} a_{ij} \theta_j \right)^2 + \lambda \| \mathbf{L}\mathbf{0} - \mathbf{d} \|_2^2
\]  

(2.30)

(b) Predict the missing data point

\[
\hat{b}_k(\lambda) = \sum_{j=1}^{m} a_{kj} \theta_j(\lambda)
\]  

(2.31)
(c) Choose the value of $\lambda$ which minimizes the cross validation function

$$V_0 = \frac{1}{n} \sum_{k=1}^{n} (b_k - \hat{b}_k(\lambda))^2 \quad (2.32)$$

The cross validation function can be further revised as (Stone 1974)

$$V_0 = \frac{1}{n} \| Q(\lambda)(A\theta(\lambda) - b) \|_2^2 \quad (2.33)$$

where

$$Q(\lambda) = \text{diag} \left( \frac{1}{1 - r_{ii}(\lambda)} \right) (i = 1, 2, ..., n) \quad (2.34)$$

and $r_{ii}$ is the $i$th element of the influence matrix

$$R(\lambda) = A(A^T A + \lambda I L^T L)^{-1} A^T \quad (2.35)$$

Similar expressions were derived by Craven and Wahba (1979) for the case of a side constraint having the standard form $\| \theta(\lambda) \|_2^2$. Golub et al (1979) showed that the ordinary cross validation method led to solutions $\theta(\lambda)$ that were rotationally dependent. They replaced $r_{ii}(\lambda)$ in Equation (2.34) with $\frac{\text{trace}(R(\lambda))}{n}$ to give the generalised cross validation (GCV) function

$$V(\lambda) = \frac{1}{n} \frac{\| (A\theta(\lambda) - b) \|_2^2}{\left( \frac{1}{n} \text{trace}(I - R(\lambda)) \right)^2} \quad (2.36)$$

which may be viewed as a weighted version of $V_0(\lambda)$.

(2) L-Curve

One way of obtaining a regularization parameter in the presence of correlated error is to define an upper bound for the side constraint and minimize the residue
\[
\min_\theta \| A\theta - b \|_2 \text{ subjected to: } \| L\theta - d \|_2 \leq \gamma
\]  
(2.37)

or alternatively to set a limit for the residue and minimize the deviation from the side constraint

\[
\min_\theta \| L\theta - d \|_2 \text{ subjected to: } \| A\theta - b \|_2 \leq \varepsilon
\]  
(2.38)

Another approach is to plot the norm of the side constraint \( \| L\theta(\lambda) - d \|_2 \) against the residue \( \| A\theta(\lambda) - b \|_2 \), obtained by minimizing the cost function in Equation (2.19) for different values of \( \lambda \). In this way, the L-curve clearly displays the compromise between minimization of side constraint and residue, which is the heart of any regularization method. The L-curve when plotted in log-log scale, normally has a characteristic L-shaped appearance (hence its name) with a distinct corner separating the vertical and the horizontal parts of the curve as shown in Figure 2.5. The vertical part of the L-curve corresponds to solutions where side constraint \( \| L\theta(\lambda) - d \|_2 \) is very sensitive to changes in the regularization parameter because the perturbation error \( \varepsilon \) from dominates \( \theta(\lambda) \) and because \( \varepsilon \) does not satisfy the discrete Picard condition. The horizontal part of the L-curve corresponds to solutions where it is the residual norm \( \| A\theta(\lambda) - b \|_2 \) that is most sensitive to the regularization parameter because \( \theta(\lambda) \) is dominated by the regularization error as long as \( b \) satisfies the discrete Picard condition. Hansen (1994) demonstrated that the norm of the side constraint is a monotonically decreasing function of the norm of the residue, and any point on the curve is a solution to the two constrained least squares problems (2.37) and (2.38). He pointed out that for a reasonable signal-to-noise ratio and satisfaction of the Picard condition the curve is approximately vertical for \( \lambda < \lambda_{opt} \), and soon becomes a horizontal line when \( \lambda > \lambda_{opt} \), with a corner near the optimal
regularization parameter $\lambda_{opt}$. Hansen and O’Leary (1993) specified $\lambda_{opt}$ as the regularization parameter with maximum curvature at the corner of the log-log plot of the L-curve. This point represents a balance between confidence in the measurements and the analyst’s intuition.

2.5 Health Monitoring

Structural health monitoring (SHM) is the process of establishing some knowledge of the current structural conditions or implementing a damage detection strategy for aerospace, civil and mechanical engineering infrastructures. The SHM process involves the observation of a system over time using periodically sampled dynamic response measurements from an array of sensors, the extraction of damage-sensitive features from these measurements, and the analysis of these features to determine the current state of system health. Prognosis process is also carried out to estimate the remaining useful life of the system by coupling information from SHM, current environmental and operational conditions. The output of long-term SHM process is periodically updated information regarding the ability of the structure to perform its intended function in light of the inevitable aging and degradation resulting from operational environment. Many investigations in the past three decades have been conducted to establish effective SHM methods and which will be briefly illustrated in the following.
2.5.1 Structural health monitoring process

The structural health monitoring can be described as a series of process such as operational evaluation, data acquisition, fusion and cleansing, feature extraction and discrimination (Farrar and Doebling 2001).

Operational evaluation relates to four aspects regarding the implementation of a structural health monitoring system: (1) to understand the economic and/or life safety motives for performing the monitoring; (2) to define the damage for the system being monitored; (3) to make clear the conditions, both operational and environmental, under which the system to be monitored functions; and (4) to illustrate the limitations on acquiring data in the operational environment. Operational evaluation defines why the monitoring is to be done and begins to tailor the monitoring to unique aspects of the system and unique features of the damage that is to be detected.

The data acquisition of the SHM process involves selecting the quantities to be measured, the types of sensors to be used, the locations where the sensors should be placed, the number of sensors, sensor resolution, bandwidth, and the data acquisition/storage/transmittal hardware. This process is a specific application in which economic considerations play a major role in these decisions. Data fusion as a discipline emerged as a result of defense organizations attempting to formalize procedures for integrating information from disparate sources (Klein, 1999). The objective is to determine battlefield situation and assess threat on the basis of data coming in from numerous different sources. The purpose of data fusion is to integrate data from a multitude of sensors with the objective of making a more robust and confident decision than is possible with any one sensor alone. Data cleansing is
the process of selectively choosing data to accept for, or reject from, the feature selection process. The data cleansing process is usually based on knowledge gained by individuals directly involved with the data acquisition.

Feature extraction and discrimination are the process of the identifying damage-sensitive properties, derived from the measured vibration responses, which allows one to distinguish between the undamaged and damaged structure. Numerous features are often identified for a structure for further damage assessment. In general, a low dimensional feature vector is desirable. Development of damage assessment model is concerned with the implementation of the algorithms that operate on the extracted features to quantify the damage state of the structure. The damage state of a system can be classified into four levels as discussed by Rytter (1993):

Level 1: Determination that damage is present in the structure;
Level 2: Determination of the geometric location of the damage;
Level 3: Quantification of the severity of the damage;
Level 4: Prediction of the remaining service life of the structure.

2.5.2 Sensor technology

The premise of vibration based SHM is that perturbations in a structural system will cause changes in measured vibration signals. Therefore, physical quantities most relevant and sensitive to the structural properties of interest should be selected for monitoring purpose. Physical quantities typically measured in response testing include accelerations, strain, and displacement. In addition, the measurements of temperature, humidity, wind, etc are required to quantify the environmental conditions of the system. Based on the physical quantities to be monitored, the type
of sensors is subsequently selected. The sensor technology has attracted significant attention in the research of SHM system in the past decade. Sensory system in SHM system has several important characteristics such as an on-board microprocessor, wireless communication, small size, and low cost. Sensing technology provides the possibility of fulfilling SHM of civil engineering structures through a densely distributed sensor network.

Westermo and Thompson (1997) discussed the development of a passive, peak strain sensor technology for SHM of bridges and buildings. The technology is based on the irreversible magnetic property changes that occur in a class of steel alloys when strained. Straser and Kiremidjian (1998) proposed a wireless modular monitoring system for SHM of civil engineering structures using smart sensor technology. The proposed network provides ease of installation, low cost, portability, and broad functionality. The sensor unit consists of a microprocessor, radio modem, data storage, and batteries. Brooks (1999) emphasized the importance of the sensor's computational capacity and defined the fourth-generation sensors as having a number of attributes: bi-directional command and data communication, all digital transmission, local digital processing, preprogramming decision algorithms, user-defined algorithms, internal self-verification and diagnosis, compensation algorithms, on-board storage and extensible sensor object models. Lynch et al (2001) demonstrated a proof-of-concept smart sensor that uses a standard integrated circuit component. Satpathi et al (1999) discussed the development of a low cost strain gauge sensor. Celebi (1999) proposed an approach by using GPS displacement measurements for SHM of structures with long periods. Shinozuka et al (2000) demonstrated the applicability of synthetic aperture radar imaging technology to the

2.5.3 Application and limitations

Lau et al (1999) discussed some aspects of the Wind and Structural Health Monitoring System (WASHMS), implemented by the Highway Department in Hong Kong to monitor the Tsing Ma, Kap Shui Mun, and Ting Kau bridges. The bridges are deemed wind-sensitive by the Highway Department. WASHMS contains the following components. The sensory system consists of 756 sensors to measure wind velocities, temperatures, accelerations, weights of vehicles, strains, and displacement. The data acquisition system consists of a personal computer controlling outstation units installed on the bridge decks. The data acquisition system collects and digitizes the signals from the sensors and transmits them via fiber optic connections to the next portion of WASHMS, the data processing and analysis system. This system contains workstations that archive, analyze, display, and record data. Finally, the processed data are forwarded to a central computer for system operation and control.

Their approach can also be used for civil engineering structures like dams, retaining walls, buildings, pavement, drainage structures, and traffic signals and signs.

Although the SHM has attracted more and more attention and many SHM systems have been developed and applied to various civil engineering structures in recent years, the currently adopted SHM techniques still hold some limitations. Friswell and Penny (1997) discussed some limitations of various SHM methods. In their opinion, the most significant limitations of SHM methods are the systematic error between a model and a structure as well as the nonstationarity of the structure. They stated that it is necessary to test any method on both simulated and real data. In conclusion, the authors give their opinion that robust identification techniques able to locate damage on realistic data sets are still a long way from reality. Aktan et al (1999) cited some critical issues for SHM, particularly of civil infrastructure, which need to be resolved for deploying reliable SHM systems. Among these issues, the authors stated that an integrated approach between academia, government, and industry is needed as well as an engineering approach that goes beyond the realm of any one branch of the engineering disciplines. The authors also stated the need for standards that govern sensor calibration and documentation of stochastic information regarding the measurement of certain environmental parameters. The authors emphasized a need for reliable standards for retrofitting structures to ensure that the retrofit accomplishes its desired task of strengthening the structures. Finally, Aktan et al (1999) questioned whether only one damage-sensitive feature or a vector of various features is suitable for practical SHM systems. They cited a number of possible causes of damage, and the inherent nonstationarity of those causes implies that both local and global approaches to SHM must be investigated simultaneously.
Figure 2.1 Flaws around welding connections (Barsom and Rolfe, 1999).

Figure 2.2 Variation of crack propagation rate with stress intensity factor (Russell, 1992).
(a)

(b)

(c)

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Figure 2.3 Configuration of some typical passive friction dampers: (a) X-braced Friction Damper (Pall and Marsh, 1982); (b) Sumitomo Friction Damper (Aiken and Kelly, 1990); (c) Energy Dissipating Restraint (Nims et al., 1993a); (d) Slotted Bolted Connection (FitzGerald et al., 1989).

Figure 2.4 Example of piezoelectric semi-active friction damper (Chen et al., 2004).
Figure 2.5 Generic form of the L-curve (Hansen, 1994).
CHAPTER 3

EVALUATION OF ATMOSPHERIC CORROSION DAMAGE TO STEEL SPACE STRUCTURES

3.1 INTRODUCTION

Steel space structures such as stadiums, gymnasiums and exhibition centers are widely constructed in China nowadays to provide special services and functions to meet the needs for social and economic development. As mentioned in Chapter 1, steel space structures may be subjected to various external loads and extreme events such as dead loads, earthquake, wind excitation, temperature effects, instability, fire, corrosion, etc, during their serviceable life span. These steel space structures, often exposed in the open air for a long time, are inevitably subjected to atmospheric corrosion. The atmospheric corrosion can change the chemical and physical properties of the steel material and weaken the strength of steel members. The degradation of steel material due to atmospheric corrosion may affect the structural durability as a whole and may cause structural damage accumulation. If the accumulated damage cannot be timely detected, the structural safety will be threatened and the damage may finally cause the partial or total collapse of the structure, resulting in economic loss and fatal casualty (Christofer, 2000). The performance assessment of steel space structures under seismic excitation, wind excitation, temperature changes and others have been studied in recent years while the corrosion-induced performance variation of steel space structures has not been
effectively investigated which will be focused in this chapter with the attention to atmospheric corrosion.

The past four decades witness the rapid development of atmospheric corrosion research in the field of material science with many field measurements and corrosion evaluation methods being proposed (Kucera, 1986; Shastry et al., 1988). For instance, the Chinese Science and Technology Committee and the National Natural Scientific Foundation of China have established a national experimental network for atmospheric corrosion of metal materials since 1983, and a huge amount of data have been collected since then. Similar activities have also been carried out in other parts of the world. Based on the data collected in the field, several empirical models have been put forward to predict the atmospheric corrosion of metal materials (Cole, 1994; Farrow and Graedel, 1996; Juan, 2003; Hou, 2004).

International concern has also increased over the past decade as it has become evident that atmospheric corrosion has resulted in substantial deterioration of buildings and structures (Cowell and Apsimon, 1996; Ninomiya et al., 1997). Ibrahim et al (1994) investigated the atmospheric corrosion of reinforcing steel and its influence on steel weight loss, strength, elongation and bending ability. Batis and Rakanta (2004) examined the performance of four different sets of reinforcing steel bars exposed to the Greek atmosphere before their installation into the concrete. Herrera et al (1995) conducted an investigation on collapse of a 10m high steel post caused by atmospheric corrosion in USA. Flavio and Stefano (2002) carried out an extensive failure analysis trying to find the reasons of the early failures of weathering steel used for many years for steel construction in the field of transportation.
A little work has yet been carried out to evaluate effects of atmospheric corrosion on structural behavior and safety of large steel space structures. This chapter first introduces the refined exponential model for estimating corrosion of steel materials at a site using a pattern recognition technique to determine the key parameters in the model. The formulae for relating structural natural frequency sensitivity to structural member thickness are then derived to assess the sensitivity of natural frequency to variation of member thickness due to atmospheric corrosion. The nonlinear static structural analysis is conducted to evaluate effects of atmospheric corrosion on the stress of structural members and the safety of steel space structures. A large steel space structure built in southern coastal area of China subjected to corrosive marine atmospheric environment has the possibility of corrosion damage after a long-term service. This structure is thereby taken as the case study to examine the feasibility of the proposed approach and to assess the potential damage caused by atmospheric corrosion to the structure.

However, not all the essential information of this large steel space structure is available for developing an integrated health monitoring and vibration control system. A relatively common reticulated shell is therefore used as a major example throughout this thesis to demonstrate the integrated health monitoring and vibration control in a systematic way. In this regard, the structural configuration of this reticulated shell is described and the evaluation of atmospheric corrosion damage to the reticulated shell is also performed in this chapter. The observations made in this chapter will be combined with those made in other chapters to provide guidelines for
the design of an integrated health monitoring and vibration control system as shown in Chapter 9.

### 3.2 Empirical Model for Predicting Atmospheric Corrosion

The study up to now indicates that there are many factors affecting the atmospheric corrosion depth of metal materials. One of the widely used empirical models for predicting atmospheric corrosion depth of metal materials is the exponential model (Feliu and Morcillo, 1993).

\[
D = AT^n
\]  
(3.1)

where \(D\) is the corrosion depth in \(\mu m\); \(T\) is the time in year; \(A\) and \(n\) are the model parameters depending on the type of metal and environmental parameters. Clearly, if the time \(T\) equals 1, the corrosion depth \(D\) equals \(A\), indicating that the parameter \(A\) represents the corrosion depth of the material in the first year.

The first-year atmospheric corrosion \(A\) is an important parameter that can be estimated based on commonly available meteorological and pollution data. After having analyzed the extensive atmospheric corrosion data collected from the seven designated locations within the national experimental network in China, Wang et al (1995) proposed the following logarithmic equation to estimate the parameter \(A\).

\[
A = C_1 \log N + C_2
\]  
(3.2)

where the coefficients \(C_1\) and \(C_2\) are the constants depending on the type of metal only; and \(N\) is the environmental index expressed as

\[
N = f_1 + f_2 + f_3
\]  
(3.3)
in which \( f_1, f_2 \) and \( f_3 \) are termed the humidity coefficient, air contamination coefficient, and rain acidity coefficient, respectively. These coefficients are expressed as

\[
f_1 = \frac{\text{hours per year for relative humidity > 80\%}}{\text{average temperature per year(\textdegree C)} \times \text{hours with sunshine per year}} \tag{3.4}
\]

\[
f_2 = (2[\text{SO}_2] + 2[\text{Cl}] + [\text{NO}_2])^{1/2} \tag{3.5}
\]

\[
f_3 = \frac{1}{2} \left( \frac{1}{10} \log \frac{\text{precipitation per year (mm/a)}}{\text{days with precipitation per year}} \right)^{7-pH/23} \tag{3.6}
\]

As a result, substituting Equation (3.2) into Equation (3.1) yields an empirical model for predicting the atmospheric corrosion depth in terms of the type of metal and the environmental index

\[
D = (C_1 \log N + C_2)T^n \tag{3.7}
\]

By applying the regression analysis to the test data collected from the national experimental network in China, the environmental index \( N \) and the corrosion development trend \( n \) in Equation (3.9) have been estimated for seven cities (see Tables 3.1 and 3.2) (Hou et al., 1994; Liang, 1998), and the constants \( C_1 \) and \( C_2 \) have also been provided for different types of metal materials (Table 3.3) (Wang et al., 1995). However, if a steel space structure is located in a city, such as Shenzhen, that is not in the national experimental network, an appropriate way should be found to estimate its environmental index \( N \) and the corrosion development trend \( n \). To this end, a pattern recognition technique is used in this study. The nine important environmental parameters in Equations (3.6) to (3.8) are first normalized for each city using the following equation.

\[
x_j = \frac{(x_j' - x_{j\text{min}}')}{(x_{j\text{max}}' - x_{j\text{min}}')}
\]  \tag{3.8}
in which $x'_j$ is the value of the $j$th environmental parameter in a city ($j=1,2,\ldots,9$ in this study); and $x'_{j\text{min}}$ and $x'_{j\text{max}}$ are the minimum and maximum values of the $j$th environmental parameter among all the cities concerned, including one with unknown $N$ and $n$.

The matching degree, $MD$, of the city with unknown $N$ and $n$ to a city with known $N$ and $n$ can then be calculated using the following equation.

$$MD(Y, Z) = 1 - \frac{1}{m} \sum_{j=1}^{m} \frac{\text{abs}[x_j(Y) - x_j(Z)]}{m}$$

(3.9)

in which $MD$ is the matching degree; $Y$ indicates the city with unknown $N$ and $n$; $Z$ refers to the city with known $N$ and $n$; and $m$ is the number of environmental parameters concerned and it is equal to 9 in this study. If there are seven cities with known $N$ and $n$, seven values of $MD$ can be calculated. The largest value of $MD$ then indicates that the corrosion development trend $n$ of the relevant city $Z$ can be used for the city $Y$. The aforementioned way makes it possible to estimate the environmental index and the corrosion development trend for the city $Y$ through several cities with known $N$ and $n$.

### 3.3 Finite Element Model of Steel Space Structures

As typical civil engineering structures, steel space structures can be analyzed based on finite element method (FEM) by adopting truss or beam elements. The displacement of a point within the $m$th element can be expressed as the function of nodal displacements

$$u^{(m)}(x, y, z) = H^{(m)}(x, y, z)\hat{U}$$

(3.10)
where \( u^{(m)} \) is the displacement vector of certain point within the \( m \)th element with coordinate \((x, y, z)\); \( H^{(m)} \) is the displacement interpolation matrix (namely shape function); \( \hat{U} \) is the nodal displacement vector.

Obtaining differential coefficients from Equation (3.10) yields
\[
\varepsilon^{(m)}(x, y, z) = B^{(m)}(x, y, z)\hat{U} \tag{3.11}
\]
where \( B^{(m)} \) is the strain-displacement matrix which includes the differential coefficients of shape function \( H^{(m)} \) to coordinates. If the initial stress is not considered, the relationship between stress \( \tau \) and strain \( \varepsilon \) of the \( m \)th element can be expressed as
\[
\tau^{(m)} = D^{(m)}\varepsilon^{(m)} \tag{3.12}
\]
where \( D^{(m)} \) is the material matrix.

The stiffness and mass matrices of the \( m \)th element can be deduced as
\[
K^{(m)} = \int_{\Omega} B^{(m)T} D^{(m)} B^{(m)} dV^{(m)} \tag{3.13}
\]
\[
M^{(m)} = \int_{\Omega} \rho^{(m)} H^{(m)T} H^{(m)} dV^{(m)} \tag{3.14}
\]
where \( \rho^{(m)} \) is the density of the \( m \)th element.

For the space structure in this study, the three dimensional (3D) beam elements with six degrees of freedoms at each node are normally adopted to simulate the static and dynamic responses under external excitations. In the local coordinate system, the nodal displacement and force of the \( m \)th element are
\[
\delta^{(m)}_e = \{u_j, v_j, w_j, \theta_{xi}, \theta_{yi}, \theta_{zi}, u_j, v_j, w_j, \theta_{yi}, \theta_{zi}, \theta_{yz}\}^T \tag{3.15}
\]
\[
f^{(m)}_e = \{U_j, V_j, W_j, M_{xi}, M_{yi}, M_{zi}, U_j, V_j, W_j, M_{yi}, M_{zi}, M_{yz}\}^T \tag{3.16}
\]
Following Equation (3.13), the stiffness matrix of 3D beam element in the local coordinate system is
Similarly based on Equation (3.14), the mass matrix of 3D beam element \(M_e^{(m)}\) can be determined. To construct the global stiffness and mass matrices, the corresponding coordinate transformation matrix \(T_u^{(m)}\) of the \(m\)th element is

\[
T_u^{(m)} = \begin{bmatrix}
\lambda & 0 & 0 & 0 \\
0 & \lambda & 0 & 0 \\
0 & 0 & \lambda & 0 \\
0 & 0 & 0 & \lambda
\end{bmatrix}
\] (3.18)

where the rotation matrix \(\lambda\) is

\[
\lambda = \begin{bmatrix}
l_{x\sigma} & l_{y\sigma} & l_{z\sigma} \\
l_{x\nu} & l_{y\nu} & l_{z\nu} \\
l_{x\pi} & l_{y\pi} & l_{z\pi}
\end{bmatrix}
\] (3.19)

in which the coefficients in Equation (3.19) represent the direction cosines of local axes in the global coordinate which can be written as follows

\[
l_{x\sigma} = \cos(x, \bar{x}), l_{y\sigma} = \cos(y, \bar{y}), l_{z\sigma} = \cos(z, \bar{z})
\] (3.20)
\[ l_{y\tau} = \cos(y, \bar{x}), \quad l_{\tau y} = \cos(y, \bar{y}), \quad l_{x\tau} = \cos(y, \bar{z}) \quad (3.21) \]

\[ l_{z\tau} = \cos(z, \bar{x}), \quad l_{\tau z} = \cos(z, \bar{y}), \quad l_{y\tau} = \cos(z, \bar{z}) \quad (3.22) \]

where, \( x, y \) and \( z \) denote the coordinate axes in the local coordinate system; \( \bar{x}, \bar{y} \) and \( \bar{z} \) denote the coordinate axes in the global coordinate system.

The element stiffness and mass matrices in the global coordinate system are expressed as

\[ \bar{\mathbf{K}}_e^{(m)} = T_a^{(m)T} \mathbf{K}_e^{(m)} T_a^{(m)} \quad (3.23) \]

\[ \bar{\mathbf{M}}_e^{(m)} = T_a^{(m)T} \mathbf{M}_e^{(m)} T_a^{(m)} \quad (3.24) \]

After determining the element stiffness and mass matrices in the global coordinate system, one can construct the position matrix of elements \( \mathbf{F}_e^{(m)} \) following the FEM connection information of each element under both local and global coordinate systems. Thus, the global stiffness and mass matrices of the space structure can be formed to establish the 3D finite element model

\[ \mathbf{K} = \sum_{m=1}^{ne} T_e^{(m)T} \bar{\mathbf{K}}_e^{(m)} T_e^{(m)} = \sum_{m=1}^{ne} T_e^{(m)T} \mathbf{K}_e^{(m)} T_e^{(m)} \quad (3.25) \]

\[ \mathbf{M} = \sum_{m=1}^{ne} T_e^{(m)T} \bar{\mathbf{M}}_e^{(m)} T_e^{(m)} = \sum_{m=1}^{ne} T_e^{(m)T} \mathbf{M}_e^{(m)} T_e^{(m)} \quad (3.26) \]

where \( ne \) is the total number of elements of the structural finite element model; \( T_e^{(m)} \) is the freedom transform matrix from element coordinate system to global coordinate system, which is the product of coordinate transformation matrix \( T_a^{(m)} \) and position matrix \( T_e^{(m)} \) of the \( m \)th element

\[ T_e^{(m)} = T_a^{(m)} T_e^{(m)} \quad (3.27) \]
3.4 SENSITIVITY OF NATURAL FREQUENCY TO ATMOSPHERIC CORROSION

A steel space structure, when exposed to natural environment under sun, rain, wind and others, is inevitably affected by atmospheric corrosion, which may result in the loss of section area of its structural members. Most of steel space structures are made of steel members with hollow section. The proper painting of the outer surface of a structural member can enhance the atmospheric corrosion resistance, but this measure can hardly be used to handle the inter surface of the structural member. The loss of cross section area or the reduction of thickness of the structural member will eventually affect the structural integrity and safety. Thus, it is necessary to put forward some ways to detect the accumulated damage due to atmospheric corrosion to a steel space structure. Since the dynamic characteristic of a steel space structure in terms of its natural frequency can be easily obtained by field measurement, the sensitivity of natural frequency to structural member thickness of a steel space structure and the relationship between the change in natural frequency and the reduction in member thickness are pursued in this study. Suppose that a finite element model is established for a steel space structure. The matrix eigenvalue problem of the steel space structure can be written as

\[ (K - \omega^2 M)\phi = 0 \]  \hspace{1cm} (3.28)

where \( M, K \) and \( \phi \) are the mass matrix, stiffness matrix and modal vector of the structure respectively. The mass matrix, stiffness matrix, and modal vector are the function of geometric properties of structural members, such as the thickness \( t_i \) of the \( i \)th structural member. Thus, the first derivative of Equation (3.28) to the thickness \( t_i \) of the \( i \)th structural member for the \( r \)th mode vibration results in
where $\omega_r$ and $\Phi_r$ are the $r$th circular natural frequency and modal vector of the structure. Since the mass and stiffness matrices are a symmetric matrix, there exists

$$\Phi_r^T(K - \omega_r^2 M)\Phi_r = [(K - \omega_r^2 M)\Phi_r]^T = 0$$

(3.30)

Thus, pre-multiplying the vector $\Phi_r^T$ to Equation (3.29) and considering Equation (3.30) yield

$$\frac{\partial \omega_r^2}{\partial t_i} = \frac{\Phi_r^T\left(\frac{\partial K}{\partial t_i} - \omega_r^2 \frac{\partial M}{\partial t_i}\right)\Phi_r}{\Phi_r^T M \Phi_r}$$

(3.31)

Equation (3.31) can be rewritten in terms of natural frequency $f_r$ in Hz

$$\frac{\partial f_r}{\partial t_i} = \frac{1}{8\pi^2 f_r} \cdot \frac{\Phi_r^T\left(\frac{\partial K}{\partial t_i} - 4\pi^2 f_r^2 \frac{\partial M}{\partial t_i}\right)\Phi_r}{\Phi_r^T M \Phi_r}$$

(3.32)

Equation (3.32) actually provides a way to calculate the sensitivity of the $r$th natural frequency to the change in thickness $t_i$ of the $i$th structural member. Assuming that the steel space structure is linear and the change in natural frequency due to the reduction of member thickness is small, the change in the $r$th natural frequency, $\Delta f_r$, due to the reduction in thickness $t_i$ of the $i$th structural member, $\Delta t_i$, can be expressed as

$$\Delta f_r = \frac{\partial f_r}{\partial t_i} \Delta t_i$$

(3.33)

It is seen that the determination of the sensitivity coefficient expressed by Equation (3.32) depends on the determination of $\partial K / \partial t_i$ and $\partial M / \partial t_i$. This can be done based on the type of cross section of a structural member and the type of element used to model the structural member. For a truss element, the element stiffness matrix and
element mass matrix are directly related to the cross section area, $A$, and the
sensitivity of stiffness matrix and mass matrix to the change in thickness $t_i$ of the $i$th
element can be expressed as

$$\frac{\partial K^e}{\partial t_i} = \frac{\partial K^e}{\partial A_i} \cdot \frac{\partial A_i}{\partial t_i}$$  (3.34)

$$\frac{\partial M^e}{\partial t_i} = \frac{\partial M^e}{\partial A_i} \cdot \frac{\partial A_i}{\partial t_i}$$  (3.35)

For an Euler-Bernoulli beam element, the sensitivity of stiffness matrix and mass
matrix to the change in thickness $t_i$ of the $i$th element can be expressed as

$$\frac{\partial K^e}{\partial t_i} = \frac{\partial K^e}{\partial A_i} \cdot \frac{\partial A_i}{\partial t_i} + \frac{\partial K^e}{\partial I_{xi}} \cdot \frac{\partial I_{xi}}{\partial t_i} + \frac{\partial K^e}{\partial I_{yi}} \cdot \frac{\partial I_{yi}}{\partial t_i} + \frac{\partial K^e}{\partial I_{zi}} \cdot \frac{\partial I_{zi}}{\partial t_i}$$  (3.36)

$$\frac{\partial M^e}{\partial t_i} = \frac{\partial M^e}{\partial A_i} \cdot \frac{\partial A_i}{\partial t_i} + \frac{\partial M^e}{\partial I_{xi}} \cdot \frac{\partial I_{xi}}{\partial t_i} + \frac{\partial M^e}{\partial I_{yi}} \cdot \frac{\partial I_{yi}}{\partial t_i} + \frac{\partial M^e}{\partial I_{zi}} \cdot \frac{\partial I_{zi}}{\partial t_i}$$  (3.37)

in which $I_{xi}, I_{yi}, I_{zi}$ are the second moment of inertia with respect to the local
coordinates $x, y, z$, respectively, of the $i$th structural member. In Equations (3.34)
to (3.37), the first derivative of either the element stiffness matrix or the element
mass matrix to the cross section area and one of the three second moments can be
easily determined based on the conventional element stiffness and mass matrices
available. For the typical cross section of a structural member, the first derivative of
the cross section area and second moment to the change in thickness can also be
determined. For space structures, the structural members of circular section and
rectangular section as shown in Figure 3.1 are often used. The section areas of
circular member and rectangular member are as follows

$$A = \pi(r_o^2 - r_i^2) = \pi t_i(r_i + r_o)$$  (3.38)

$$A = b^o h^o - b^i h^i$$  (3.39)
in which \( r_i \) and \( r_o \) are the inner and outer radius of circular section respectively; \( b_i \) and \( h_i \) are the width and height of inner rectangular section; \( b_o \) and \( h_o \) are the width and height of outer rectangular section.

By omitting the second order and above of the change in thickness \( \Delta t \), the aforementioned first derivatives to the thickness change caused by the reduction of inner surface of the circular cross section member can be derived as

\[
\frac{\partial A}{\partial t_i} = 2\pi r_i \tag{3.40}
\]

\[
\frac{\partial I_y}{\partial t_i} = \frac{\partial I_z}{\partial t_i} = \pi r_i^3 \tag{3.41}
\]

\[
\frac{\partial I_z}{\partial t_i} = 2\pi r_i^3 \tag{3.42}
\]

in which \( r_i \) is the inner radius of circular cross section of the structural member as shown in Figure 3.1 (a). The first derivatives to the thickness change caused by the reduction of outer surface of the circular cross section member can be given as

\[
\frac{\partial A}{\partial t_o} = 2\pi r_o \tag{3.43}
\]

\[
\frac{\partial I_y}{\partial t_o} = \frac{\partial I_z}{\partial t_o} = \pi r_o^3 \tag{3.44}
\]

\[
\frac{\partial I_z}{\partial t_o} = 2\pi r_o^3 \tag{3.45}
\]

in which \( r_o \) is the outer radius of circular cross section of the structural member. For the rectangular cross section member as shown in Figure 3.1 (b), the sensitivity coefficients with respect to the loss of thickness on the inner surface are

\[
\frac{\partial A}{\partial t_i} = 2b_i + 2h_i \tag{3.46}
\]

\[
\frac{\partial I_z}{\partial t_i} = \frac{1}{6}(b_i + h_i)^3 \tag{3.47}
\]
\[ \frac{\partial I_y}{\partial t^{ou}} = \frac{(h^{ou})^2}{6} (3b^{ou} + h^{ou}) \] (3.48)

\[ \frac{\partial I_z}{\partial t^{ou}} = \frac{(b^{ou})^2}{6} (3h^{ou} + b^{ou}) \] (3.49)

The sensitivity coefficients with respect to the loss of thickness on the outer surface of the structural member are

\[ \frac{\partial A}{\partial t^{ou}} = 2b^{ou} + 2h^{ou} \] (3.50)

\[ \frac{\partial I_x}{\partial t^{ou}} = \frac{1}{6}(b^{ou} + h^{ou})^3 \] (3.51)

\[ \frac{\partial I_y}{\partial t^{ou}} = \frac{(h^{ou})^2}{6} (3b^{ou} + h^{ou}) \] (3.52)

\[ \frac{\partial I_z}{\partial t^{ou}} = \frac{(b^{ou})^2}{6} (3h^{ou} + b^{ou}) \] (3.53)

Once all the sensitivity coefficients of the element stiffness and mass matrices with respect to the thickness change are available, the assembly of them in accordance with the conventional finite element method yields the required matrices \( \frac{\partial K}{\partial t_i} \) and \( \frac{\partial M}{\partial t_i} \), by which the change in the \( r \)th natural frequency, \( \Delta f_r \), due to the reduction in thickness \( t_i \) of the \( i \)th structural member, \( \Delta t_i \), can be determined according to Equation (3.33).

### 3.5 Stress Changes Due to Atmospheric Corrosion

The reduction of cross section thickness of all the structural members in a steel space structure due to atmospheric corrosion will cause stress changes, which will in turn affect the safety of the structure. The gravity forces of structural members, roof slabs and other auxiliary components are taken as dead loads acting on the structure. In
considering that large scale steel space structure may experience large deformation, a nonlinear static analysis is conducted in this study to assess the stress changes in the structure under dead loads. As opposed to the linear static analysis, the nonlinear static analysis is based on the deformed structure, and the deformation and stress distribution of the structure in the final stage can only be determined through numerical iterations. Both the geometric nonlinearity and material nonlinearity are considered in the nonlinear analysis. The Newton-Raphson iteration method is used in this study, which makes the solution converge at each load increment to ensure the accuracy of the final computed results. The flow chart for the nonlinear static analysis of a large steel space structure due to atmospheric corrosion under dead loads is shown in Figure 3.2. The environmental parameters for the location where the structure is built should be input first, from which the environmental index and other parameters in Equation (3.7) can be estimated. The reduction of thickness of each member of the structure can then be calculated and input to compute the actual stiffness of the structural members. Finally, a standard nonlinear static analysis should be conducted. In Figure 3.2, $k_T$ and $k_T^e$ are the tangent stiffness matrix of an element in the global and local coordinate, respectively. The matrix $K_T$ is the total tangent stiffness matrix of the structure. $X$, $U$, and $\Delta U$ are the nodal coordinate vector, displacement vector, and displacement increment vector, respectively, in the global coordinate. $u$ and $\Delta u$ are the displacement vector and displacement increment vector, respectively, in the local coordinate. $r^e$ and $\Delta r^e$ are the element force vector and element force increment vector, respectively, in the local coordinate. $r$ and $R$ are the element force vector and total internal force vector, respectively, in the global coordinate. $F$ is the external force vector, and $\Delta F$ is the unbalanced force vector.
is the transformation matrix from the global coordinate to the local coordinate for an element.

3.6 APPLICATION TO A LARGE STEEL SPACE STRUCTURE

The steel space structure built in coastal area subjected to corrosive marine atmospheric environment is susceptible to corrosion damage after a long-term service. A large steel space structure is therefore taken as the case study to examine the feasibility of the proposed approach and to assess the potential damage caused by atmospheric corrosion to the structure.

3.6.1 Description of a large steel space structure

Figures 3.3 and 3.4 display the front view and bird’s eye view of the large steel space structure consisting of three separate parts to provide roof function in order to form a well-integrated office complex as designed by the architect. The office complex is located in Shenzhen, a southern coastal city of China adjacent to Hong Kong (Qu et al 2004). The middle part of the roof is the major force bearing frames of the whole roof. The length of the middle part of the steel space structure is 258\(m\) in the east-west direction, and its width in the north-south direction varies from 130\(m\) at the two ends to 120\(m\) at the middle of the structure (see Figures 3.5 and 3.6). The space structure possesses a curved elevation with the lowest level about 36\(m\) above the ground at the two ends and the highest level about 73\(m\) above the ground at the middle. The thickness of the space structure in the elevation varies from 4.47\(m\) at the two ends to 8.79\(m\) at the middle. The space structure is constructed with a triple-layer grid. The top layer, the middle layer, and the bottom layer of the space structure
are formed as a square grid as shown in Figures 3.7 (a), 3.7 (c), and 3.7 (e), respectively, with two openings left for the connections with the circular tower and the rectangular tower through two huge truss girders. The top layer and the bottom layer are connected to the middle layer through the top brace system shown in Figure 3.7 (b) and the bottom brace system shown in Figure 3.7 (d) with all spherical joints welded. The two huge truss girders, as shown in Figure 3.7 (f), are connected to the triple-layer grid at the locations of the circular tower and the rectangular tower to form a complete space structure. One truss girder then sits on the circular tower through eight spherical bearings that are movable in all directions, and the other truss girder sits on the rectangular tower through four spherical bearings. The space structure also sits on the east building and the west building at its two ends with a total of eight supports, each of which is formed using 16 members arranged in shape of a tree and connected to a spherical bearing, as shown in Figure 3.7 (g). A 3D finite element model is established for the steel space structure using a commercial computer package (ANSYS). The model has a total of 12499 beam elements and 2921 nodes with 6 degrees of freedom at each node. All the joints in the finite element model are assumed to be rigid. Each spherical bearing is modeled as two springs arranged in the $x$- and $y$- direction respectively. The spring stiffness is about $9.14 \times 10^6$ N/m which is provided by the designers of the structure. The movement of all the supports in the vertical direction is restricted. For the sake of convenience in the subsequent discussion, the beam elements in each component of the steel space structure are numbered differently. The beam elements in the eight tree-shaped supports are numbered from 1 to 128 (denote zcg), the elements in the bottom layer are numbered from 129 to 2086 (denote xxg), the elements in the bottom brace system are counted from 2087 to 5315 (denote xfg), the elements in the middle layer
are counted from 5316 to 6938 (denote zxg), the elements in the top brace system are counted from 6939 to 10180 (denote sfg), the elements in the top layer are counted from 10181 to 11810 (denote sxg), and the elements in the two truss girders are counted from 11811 to 12499 (denote hjt). The number of elements in the eight three-shaped supports is only 1% of the total number of elements used in the structure while the number of elements in the two truss girders is 5.5% of the total number of elements used in the structure. The structural members used in the eight three-shaped supports and the two truss girders are made of Q345A steel (16Mn) with a yielding stress of 345MPa. The structural members used in the three layers and the two brace systems are made of Q235B steel (A3) of a yielding stress of 235MPa. Except that the main chords in the two truss girders are of hollow rectangular section, all the other members are of hollow circular section.

3.6.2 Dynamic characteristics and stress levels without corrosion

The dynamic characteristics analysis is conducted based on the established finite element model of the steel space structure. The masses of all the structural members, roof slabs and other auxiliary components are considered to construct the mass matrix. The first 10 vibration modes of the structure are depicted in Figure 3.8. The first vibration mode is a global vibration mode mainly due to the movement of spring supports in the y-direction. The second vibration mode is also a global vibration mode but mainly due to the movement of spring supports in the x-direction. The third vibration mode is a global torsional vibration mode in the x-y plane due to the movement of spring supports. The fourth to the tenth vibration modes all appear mainly in the vertical direction because of the movements of the two truss girders in different combinations. The first 10 natural frequencies computed indicate that the
natural frequencies of the structure are closely spaced. The field measurement of vibration modes of the structure may be a difficult task.

The nonlinear static analysis of the space structure under dead loads is also carried out to find stress levels of all the structural members. The gravity forces of all the structural components, roof slabs and other auxiliary components are taken as the dead loads acting on the structure. Figure 3.9 shows the percent ratio of working stress to yielding stress of all the structural members. The maximum stress level is less than 80%. The number of structural members with the stress ratio less than 10% accounts for 73% of the total structural members, and the 99.5% structural members have a stress ratio less than 30%. Only two members have a stress level between 70% and 80%. In general, the structural members in the two truss girders and the two brace systems have higher stress level than those in the eight tree-shaped supports and the three layers. It is noted that the stress levels calculated here consider dead loads only.

3.6.3 Atmospheric corrosion of materials

The empirical model presented for atmospheric corrosion is now applied to predict the atmospheric corrosion depth of the steel members of the space structure. Figure 3.10 shows the relative locations of Shenzhen and other seven cities. The national experimental network has obtained the environmental index $N$ and the atmospheric corrosion trend $n$ for the seven cities but not for Shenzhen. Therefore, the environmental parameters of Shenzhen are collected and listed in Table 3.1. The environmental index $N$ of Shenzhen is calculated as 1.113 using Equation (3.3). This value is similar to that of Guangzhou and Wanning. To determine the atmospheric
corrosion trend $n$ for Shenzhen, the pattern recognition technique expressed by Equations (3.8) and (3.9) is used. The seven matching degrees are depicted in Figure 3.11 for the seven cities. The best match to Shenzhen is Guangzhou, which is a city nearest to Shenzhen (see Figure 3.10). Both cities are affected by industrial atmosphere, and the contamination contents in Shenzhen are close to Guangzhou. Thus, it is reasonable to use the coefficient $n$ of Guangzhou for Shenzhen, that is, $n=0.48$ for Q345A steel and $n=0.45$ for Q235B steel according to Table 3.2. The constants $C_1$ and $C_2$ are 39.55 and 65.94 for Q345A steel, and 39.36 and 61.34 for Q235B steel according to Table 3.3.

With all the model parameters determined, the corrosion depth of steel members in the space structure can be predicted using Equation (3.7). Figure 3.12 shows the variation of corrosion depth of Q235B steel and Q345A steel with time in year. Figure 3.13 displays the variation of corrosion rate with time in year. It is seen that the corrosion depth of the two materials increases with time. Q235B steel has relatively smaller corrosion depth than Q345A steel. The atmospheric corrosion rate of the material depth is almost the same for the two materials. It is clear that the corrosion rate of material depth is much faster in the first 5 years than later.

The variation of corrosion depth of steel members with atmospheric contaminant contents SO$_2$, Cl$^-$ and NO$_2$ is displayed in Figures 3.14, 3.15 and 3.16 respectively. It can be concluded from these figures that the influence of contaminants on the steel atmospheric corrosion is obvious and the increase of SO$_2$, Cl$^-$ and NO$_2$ content in the atmosphere may cause the increase of corrosion depth. Following the results of Figure 3.14, the variation of corrosion depth caused by SO$_2$ deposit can be divided
basically into two stages: the corrosion depth develops rapidly if the deposit is less than 0.3 (mg/100cm² d); after that the developing velocity reduces to some extent. The variation of Cl⁻ deposit with time presents the same trend with SO₂ as plotted in Figure 3.15. The curves in Figure 3.16 demonstrate that the variation of corrosion depth with NO₂ content does not show obviously two phases. To summarize the variation of corrosion depth with the three major contaminations, one can conclude that the influence of SO₂ and Cl⁻ is stronger than that of NO₂. The comparison between the two different steel materials reveals that the corrosion depth of Q345 steel is slightly larger than that of Q235 steel while their increasing trends are quite similar.

Figures 3.17, 3.18 and 3.19 demonstrate the variation of corrosion depth with hours of average relative humidity beyond 80%, annual average temperature and sunshine hours respectively. The curves of the above three figures indicate that the variation of climatic factors has influence on the atmospheric corrosion to a small extent which is much less than that of air contaminants. The results in Figure 3.17 clearly demonstrate that corrosion depth increases with the annual average relative humidity (>80%) while the increment rate is quite small. However, the effects of annual average temperature and sunshine hours on corrosion depth are quite different. The curves in Figures 3.18 and 3.19 indicate that the corrosion depths of Q345 and Q235 steel reduce slowly following the increase of annual average temperature and sunlight hours. After exposed in the open air for certain periods, some corrosive outcomes will be produced on steel surface. As far as Q345 and Q235 steel are concerned, their differences of corrosion depth under various average relative humidity, average temperature and sunshine hours are quite stable which is similar to
those of atmospheric contaminant contents. These results further indicate that the steel materials used in the example structure basically have the same corrosion trend despite their slight difference in chemical components. Furthermore, the variation of corrosion depth with precipitation, precipitation days and PH value of rain is also investigated and the corresponding results demonstrate their slight influence on corrosion depth compared to the aforementioned six factors.

3.6.4 Effects of atmospheric corrosion on natural frequencies

The steel space structure concerned is made of steel members of either circular or rectangular hollow section. The outer surfaces of all the structural members are painted to prevent atmospheric corrosion, but the inner surfaces of all the structural members are not painted because of operation inconvenience. To this end, the atmospheric corrosion on the inner surfaces of the structural members should be considered. Nevertheless, the atmospheric corrosion on both the inner and outer surfaces is also considered as an extreme case and compared with the case of inner surface corrosion only. For either case, the sensitivity of the \( r \)th natural frequency to the change in thickness \( t_i \) of the \( i \)th structural member is first computed using Equation (3.32). The change in the \( r \)th natural frequency, \( \Delta f_r \), due to the reduction in thickness \( t_i \) of the \( i \)th structural member, \( \Delta t_i \), is then computed using Equation (3.33). Finally, the algebraic summation of the changes in the \( r \)th natural frequency due to the reductions in thickness of all the structural members gives the final change in the \( r \)th natural frequency, from which the effect of atmospheric corrosion on the \( r \)th natural frequency can be assessed.
Figure 3.20 shows the sensitivities of the first 10 natural frequencies to the thickness change of each member due to inner surface corrosion. It is seen that the first three natural frequencies are more sensitive to the thickness change of members in the eight tree-shaped supports than structural units. All the higher natural frequencies are more sensitive to the thickness change of members in the two truss girders than other structural units. This is consistent with the structural configuration and the modes of vibration: the first three modes of vibration are mainly due to the movement of the supports while the higher modes of vibration are mainly due to the movement of the two truss girders. It is noted that the first and sixth natural frequencies are also sensitive to the thickness change of some of members in the two brace systems. However, the thickness changes of structural members in the middle layer of the structure due to corrosion affect the natural frequencies only slightly despite their important connection functions between the top and bottom layers.

The changes in natural frequencies of the structure due to inner surface corrosion are displayed in Figure 3.21 for the corrosion period of 1, 4, 10 and 20 years, respectively. The changes in the first 5 natural frequencies with corrosion time are shown in Figure 3.22. It is seen that as corrosion years increase, the changes in natural frequencies also increase. The changes in the first 5 natural frequencies are negative, indicating that the natural frequencies are reduced. The second natural frequency of the structure has the maximum change due to inner surface corrosion. The changes in some of higher natural frequencies are positive, implying that these natural frequencies are actually increased. This is because while the reduction of member thickness due to corrosion causes the stiffness reduction, it also causes the mass reduction. Whether the change in a natural frequency is positive or negative
depends on the relative extent of the stiffness reduction effects and the mass reduction effect.

It is seen that the second natural frequency of the structure has the maximum change due to inner surface corrosion. The maximum change in the second natural frequency due to inner surface corrosion is about 1% in 20 years. Even though both the inner and outer surface corrosions are considered, the maximum change in the second natural frequency is about 2% in 20 years. Therefore, it can be concluded that the natural frequencies of the large steel space structure considered in this study are only slightly affected by the atmospheric corrosion of materials. However, this conclusion may not be applicable to other steel space structures in other places.

3.6.5 Effects of atmospheric corrosion on member stresses

The effects of atmospheric corrosion on member stresses of the structure under dead loads are investigated in this section. The atmospheric corrosion will cause the reduction of cross section areas of the structural members, and therefore it will also cause the reduction of gravity forces and structural stiffness. Such changes will in turn affect the level and distribution of structural member stresses. On the other hand, the gravity forces of other structural components, such as joints and cladding, are regarded as constant forces acting on the structural nodes without changes. Nonlinear static analyses of the structure are then carried out according to the computation algorithm shown in Figure 3.2 for 10, 20, and 50-year atmospheric corrosion.

Figure 3.23 shows the statistics of structural members with various levels of stress change under 10, 20 and 50-year atmospheric corrosion. The results shown in Figure
3.23 (a) are for inner corrosion only while the results displayed in Figure 3.23 (b) are for double surface corrosion. It is seen from Figure 3.23 (a) that 94.1%, 91.9% and 87.4% of the structural members have a stress change no more than ±10% for 10, 20 and 50 year-atmospheric corrosion, respectively. The structural members having a stress change large than ±100% are 0.52%, 0.78% and 1.21% of the total structural members for 10, 20, and 50 year-atmospheric corrosion, respectively. It is clear that with the increase of atmospheric corrosion year, the numbers of structural members with large stress change increase. There exist both negative and positive stress changes, indicating stress redistribution in the structure. Nevertheless, the structural members of increasing stress are more than those of decreasing stress. Similar observations can be made for the structural members with double surface corrosion, as shown in Figure 3.23 (b). Compared with the structural members of inner surface corrosion only, the numbers of structural members of large stress change significantly increase while the numbers of structural members of small stress change considerably decrease when double surface corrosion is considered.

To further illustrate the stress change and redistribution, the stress changes in 128 structural members of the eight tree-shaped supports are plotted in Figure 3.24 (a) for inner surface corrosion only and Figure 3.24 (b) for double surface corrosion. It is seen that though the stresses in most of the structural members are only slightly affected by atmospheric corrosion, a few of structural members have large stress change and such a stress change increases with increasing corrosion year. For a 50-year atmospheric corrosion, the maximum stress change reaches 93% for inner surface corrosion and it reaches about 200% for double surface corrosion. Though the stress changes in the structural members due to double surface corrosion are
much larger than those due to inner surface corrosion, the structural members of large stress changes remain almost in the same positions.

A careful examination of stress changes in all the structural members reveals that for the concerned structure, the structural members of higher stress level have small stress change ratio only while large stress changes occur in the structural members of lower stress level. Listed in Table 3.4 are the stress changes in the two structural members of the highest stress level due to inner surface corrosion. The two members belong to the top brace system. It is seen that the stress change ratio increases with increasing corrosion year but the stress change is small and less than 4% even with a 50-year corrosion. However, such results may not be applicable to other structures in other places.

3.7 APPLICATION TO RETICULATED SHELL

As mentioned before, the research in this thesis aims to develop and establish the framework for the conceptual design of integrated control and monitoring system. To this end, a real reticulated shell is adopted throughout this thesis to achieve this objective. The structural performance under various excitations and extreme events such as dead loads, earthquake, wind excitation, temperature change, instability, fire, corrosion, etc, are examined in different chapters to provide guidelines for the design of integrated system. In this chapter, the structural configuration of the example reticulated shell is first introduced. Then the finite element mode of original structure is constructed using three dimensional beam elements. The static analysis under dead loads and modal analysis of the reticulated shell are carried out. The nodal displacement and member stresses under dead loads are computed to assess the
structural performance. The dynamic characteristics of the example shell are also analyzed to explore its vibration features. Then the atmospheric corrosion damage to the reticulated shell is investigated to evaluate its safety states under corrosive environment. The observations made in this chapter will then be taken into consideration in the design of integrated monitoring and control system in Chapter 9.

3.7.1 Structural description

Large spans have always fascinated architects and engineers. Reticulated shells provide an easy and economical method of roofing large areas and are used frequently by the designers who realize the advantages and the impressive beauty of this form of construction. Sports stadia, assembly halls, exhibition centres, swimming pools, shopping arcades and industrial buildings are typical examples of structures where large unobstructed areas are essential. Space shells are of special interest in this respect, as they enclose a maximum amount of space with a minimum surface and have proved to be very economical in terms of consumption of constructional materials (Makowski 1984). The recent development of space structures shows that remarkable progress in the field of reticulated shell has been achieved during the past three decades in many countries. Various new types of reticulated shells have been developed across the world. People now realize the full structural potential of this shell, using them extensively for commercial and industrial structures.

Steel was first used in shell construction in 1811, when Belanger and Brunet covered the central part of the Corn Market in Paris with a steel roof. Much interest was aroused among the engineers and architects by this construction project though it was
not much more than an adaptation of the methods of timber construction to the use of wrought iron. The first reinforced concrete (RC) shell, invented by Dr. Walter Bauersfeld, was built in 1922. It was the world’s first light-weight structural framework, which was covered with cement, creating the first thin-shell concrete structure in the history of civil engineering. However, the RC shells requiring elaborate and very expensive false work were slow in construction and often not really economical in terms of cost. This was the time when the advantages of steel and aluminum material were appreciated. The past thirty years has been specially marked all over the world through the bold and imaginative use of metal shells. Reticulated shells are also exceptionally suitable for covering large space structures in which large unobstructed areas are required. As far as the remarkable reticulated shell is concerned, there is no doubt that the Astrodome in Texas, having a span of 200\(m\) (See Figure 3.25) and New Orleans Superdome, having a span of 213\(m\), receives our greatest attention. These reticulated shells devised by Dr. Kiewitt, clearly shows the superiority in pursuing large unobstructed areas (Makowski 1984). Another example is the Makomanai shell over the Sapporo Olympic Arena, Japan. This shell has a clear span of 105\(m\) and was designed and built by Tomoegumi Ironworks Ltd., Tokyo. Similarly, many other reticulated shells are constructed across the world to meet various social and economic requirements (See Figure 3.27).

Owing to the wide application of reticulated shells in recent years, a Kiewitt-type reticulated shell with the similar configuration of Astrodome in Texas (See Figure 3.25), constructed in Shijiazhuang City, Northern China having a span of 65\(m\) and a height of 9.916\(m\), is taken as a case example in this thesis to establish the framework and carry out the conceptual design for the integrated health monitoring and health
monitored system. This reticulated shell consists of 169 rigid nodes and 456 steel members. The length of structural members varies from 4 to 6 meter. Each member is constructed using thin-wall steel tube having a wall thickness of 5mm. The outer diameter and cross section area for all the radial and circular members are 180mm and 0.002749$m^2$ respectively. The outer diameter and cross section area for all the skew members are 159mm and 0.002419$m^2$ respectively. The Young’s modulus for the steel material is $2.07 \times 10^7$MPa. The yielding strength of the steel material is 235MPa. The density and Poisson’s ratio of material are 7800kg and 0.28 respectively. The reticulated shell is covered with roof slabs having an equivalent mass of 20$kg/m^2$. The finite element model of this reticulated shell is constructed by using 3 dimensional beam elements. The mass matrix includes the contribution of structural members and other components such as roof slabs. Figures 3.28 (a) and (b) display the finite element model of example reticulated shell which has 169 nodes and 456 spatial beam elements respectively. The reticulated shell consists of 48 radial members (No. 1 to 48), 168 circular members (No. 49 to 216) and 240 skew members (No. 217 to 456). The 48 radial members radiate from the shell vertex (Node 1) in eight directions respectively. The 168 circular members are located on 6 concentric circles, counted from the smallest circle 1 to the largest circle 6, with node 1 being the centre. All the nodes on the outermost circle (circle 6) are rigidly constrained based on design configuration.

3.7.2 Static responses

The gravity forces of steel members and all the other components such as roof slabs are taken as dead loads acting on the reticulated shell to examine its performance.
When subjected to dead loads, the force equation of the concerned shell is expressed as

\[ \mathbf{KX} = \mathbf{F} \quad (3.54) \]

where \( \mathbf{K} \) is the global stiffness matrix of the shell; \( \mathbf{F} \) is the static loads acting on the shell; \( \mathbf{X} \) is the static deformation vector of the structural nodes. The static deformation of the reticulated shell is computed and plotted in Figures 3.29 (a) and (b). Clearly owing to the symmetric distribution of static loads, the nodal displacement of concerned shell is also symmetric in essence. The deformation of all the nodes under static loads displayed in Figure 3.29 (b) clearly reveals that each node has translation in three directions due to the element deformation. The nodal vertical displacement is much larger than those of other two horizontal directions. The further numerical investigation reveals that the maximum vertical displacement of the concerned shell is 0.659 cm which is about 1/7211 of the shell span. Some properties of shell static displacement can be summarized as: (1) the nodes on the first circle are located on the symmetric axes of the shell and thereby the displacement is the same; (2) the vertical displacement of nodes on circle 2, 3 and 4 is close to some extent. The displacement of nodes on radial members is slightly larger than that of nodes on skew members.

Figures 3.30 (a)-(f) display the magnitude and spatial distribution of axial force, shear force and bending moment of all the structural members. By comparing the responses in detail, one can find that the axial force responses of reticulated shell under static loads are quite larger than those of shear force and bending moment. Figure 3.30 (a) demonstrates that all the members are subjected to compressive forces under the current gravity loads. The axial forces of radial members are quite
larger than those of skew members and the axial forces of radial members among circle 2 and 4 are larger than those of radial members among circle 4 and 6. Similar observations can be made for the axial forces of circular members as shown in Figure 3.30 (a). The axial forces of circular members on circular 2, 3 and 4 are larger than those of circular members on the other circles. The magnitude and distribution of stress and axial deformation of all the structural members under gravity loads are plotted in Figures 3.31 (a) to (f). The distributions of maximum stress and maximum axial stress are demonstrated in Figures 3.31 (a) and (b) respectively. Similar to the observations made from internal forces, the member stresses are mainly contributed by axial stress. The stress distributions of circular and radial members shown in Figures 3.31 (c) and (d) clearly reveal that the radial and circular members among circle 1 and circle 4 are the major force-bearing members. The results of Figure 3.31 (e) illustrate that the skew members with relatively larger stresses is adjacent to the radial members. The observations from member axial deformation distribution plotted in Figure 3.31 (f) demonstrate that the axial deformation of radial members is obviously larger than those of circular members.

In real application, the boundary constraints of reticulated shells can be taken as joint constraints or rigid constraints according to actual construction conditions. Although the concerned shell is rigid boundary following the design requirements, the rigid boundary may damage after a long-term service due to corrosion, concrete creep and bearing crack. The static responses of the reticulated shell with joint boundary conditions are therefore computed to understand its force-bearing performance. The nodal displacement and member axial stresses under both rigid and joint boundary conditions are plotted in Figures 3.32 (a) and (b) respectively. Obviously, the nodal
displacement and member axial stress under two boundary conditions are quite close which demonstrates that the structural static responses at boundary nodes are not rotation but translation.

3.7.3 Modal properties

The first 50 natural frequencies of the shell are listed in Table 3.5 and the corresponding first 20 mode shapes are plotted in Appendix A. Clearly, the natural frequencies of the reticulated shell are very close and there exists many duplicate natural frequencies which relate to the symmetric mode shapes. The ratio of height to span \( (f/L) \) is 1:6.54 which is relatively small. The mode shapes demonstrate the coupled vibration in both horizontal and vertical directions. Owing to the symmetric axes of the shell, there exist many symmetric mode shapes such as the 1st and 2nd, 3rd and 4th, 5th and 6th mode shapes, etc. The parameter study is carried out in this section to investigate the effects of various factors on the dynamic features which includes span \( (L) \), ratio of height to span \( (f/L) \) and boundary constraint. The values of various factors are as follows

(1) Span: 45\( m \), 55\( m \), 64.8\( m \), 75\( m \);

(2) Ratio of height to span: 1/8, 1/6.54, 1/5, 1/4;

(3) Boundary constraint: rigid and joint constraints.

The variation of the first 50 natural frequencies with spans plotted in Figure 3.33 reveals that the obvious effects of span on shell dynamic properties. By keeping the shell height and increasing the span, the vibration models of mode shapes gradually transfer from vertical vibration to coupled vibration in both horizontal and vertical directions. Simultaneously, the magnitude of natural frequencies substantially reduces. The variation of the first 50 natural frequencies with \( f/L \) ratio is plotted in
Figure 3.34 which provides similar conclusions to those in Figure 3.33. If the shell height is kept unchanged and simultaneously reduces the $f/L$ ratio, the shell configuration will be transferred from a 3D spatial structure to a 2D plane structure. The natural frequencies will be gradually reduced. The effects of boundary conditions are analyzed and provided in Figure 3.35. Clearly, the difference of natural frequencies and mode shapes between rigid and joint constraints is quite small.

### 3.7.4 Evaluation of atmospheric corrosion damage

Similar to the analytical procedure of the large steel space structure in Shenzhen, the evaluation of atmospheric corrosion on the reticulated shell is also carried out. The environmental index $N$ of Shijiazhuang City is calculated as 1.9336 using Equation (3.3) and this value is similar to that of Qingdao. To determine the atmospheric corrosion trend $n$ for steel material used in Shijiazhuang City, the pattern recognition technique expressed by Equations (3.8) and (3.9) is used. The seven matching degrees are depicted in Figure 3.36 for the seven cities. The best match to Shijiazhuang is Qingdao. Thus, it is suggested to use the coefficient $n$ of Qingdao for Shijiazhuang, that is, $n=0.44$ for Q235B steel according to Table 3.2. The constants $C_1$ and $C_2$ are 39.36 and 61.34 for Q235B steel according to Table 3.3. With all the model parameters determined, the corrosion depth of steel members in the reticulated shell can be predicted using Equation (3.7). Figure 3.37 and Figure 3.38 display the variation of corrosion depth and corrosion rate of steel material with time in year respectively. Similar to the conclusions obtained from the larger steel space structure in Shenzhen, the corrosion depth of steel material increases with time and the corrosion rate of material depth is much faster in the first 5 years than later.
Figure 3.39 shows the sensitivities of the first 8 natural frequencies to the thickness change of each member due to inner and outer surface corrosion respectively. Clearly, the member distributions of frequency sensitivity for inner and outer corrosion are the same for the first 8 natural frequencies while the sensitivity magnitude of inner corrosion is slightly larger than those of outer corrosion. Further examination on sensitivity distribution reveals that the structural members with large sensitivity are radial members and circular members. The first two natural frequencies are more sensitive to the thickness change of radial members and some skew members within the first two circles. The third and fourth natural frequencies are more sensitive to the thickness change of circular members in the three and four circles. While for the other higher natural frequencies, the sensitive members are the radial members and skew members parallel to radial members. To conclude the member distribution with large frequency sensitivity to member thickness, one can found that the radial members are most sensitive to section loss caused by atmospheric corrosion. Simultaneously, some circular member and skew member close to the radial members are also sensitive to section loss.

The changes in natural frequencies of the structure due to inner and double surface corrosion are displayed in Figure 3.40 for the corrosion period of 1, 4, 10 and 20 years, respectively. The changes in the first 5 natural frequencies with corrosion time are shown in Figure 3.41. Clearly, the changes in natural frequencies increase as corrosion year increases. The changes in all the first 20 natural frequencies are negative, indicating that the natural frequencies are reduced. This is because the reduction of member thickness due to corrosion causes the stiffness reduction though
it also causes the mass reduction. The change in a natural frequency is negative mainly because the relative extent of the stiffness reduction effects is larger than those of mass reduction effects. It is seen that the 16th natural frequency of the structure has the maximum change due to both inner and double surface corrosion. The maximum change in the 16th natural frequency due to inner surface corrosion is about 0.1% in 20 years. Even though both the inner and outer surface corrosions are considered, the maximum change in the second natural frequency is about 1.4% in 20 years. Compared with the results of the aforementioned large steel space structure, it can be concluded that the effects of atmospheric corrosion on natural frequencies of the large steel space structure are larger than those of reticulated shell.

The effects of atmospheric corrosion on member stresses of the reticulated shell under static loads are also investigated in this section. Nonlinear static analyses of the shell are carried out for 10, 20, and 50-year atmospheric corrosion and the variation of maximum stress of members are provided in Figure 3.42 under double surface corrosion. It is seen from Figure 3.42 (a) that the maximum stress changes under 10, 20 and 50 year-atmospheric corrosion are 0.11\(MPa\), 0.14\(MPa\) and 0.21\(MPa\) respectively. It is clear that with the increase of atmospheric corrosion year, the variation of maximum member stress also increases while this change is quite small compared to member’s ultimate stress. There exist both negative and positive stress changes, indicating stress redistribution in the structure. Nevertheless, the structural members of increasing stress are more than those of decreasing stress. Similar observations can be made for the structural members with double surface corrosion.
3.8 SUMMARY

A framework for evaluation of potential damage due to atmospheric corrosion to steel space structures has been established and applied to a large steel space structure and a reticulated shell built in China respectively. The case study demonstrates the feasibility of the proposed framework. For the large steel space structure, the nonlinear static analysis of the steel space structure under dead loads is carried out to examine stress levels of all the structural members. The number of structural members with the stress ratio less than 10% accounts for 73% of the total structural members, and the 99.5% structural members have a stress ratio less than 30%. The structural members in the two truss girders and the two brace systems have higher stress level than those in the eight tree-shaped supports and the three layers. The corrosion depth of the two materials increases with time. Q235B steel has relatively smaller corrosion depth than Q345A steel. The atmospheric corrosion rate of the material depth is almost the same for the two materials and the corrosion rate of material depth is much faster in the first 5 years than later. The case study also shows that the changes in natural frequencies of the steel space structure due to either inner surface corrosion or double surface corrosion are very small. The second natural frequency of the structure has the maximum change due to inner surface corrosion which is about 1% in 20 years. Even though both the inner and outer surface corosions are considered, the maximum change in the second natural frequency is about 2% in 20 years. Therefore, it can be concluded that the natural frequencies of the large steel space structure considered in this study are only slightly affected by the atmospheric corrosion of materials. Though the stresses in most of the structural members are only slightly affected by atmospheric corrosion, a few of structural
members have large stress change and such a stress change increases with increasing corrosion year.

While both the natural frequency and stress changes of reticulated shell due to atmospheric corrosion are quite small. These observations are different from the results of the aforementioned large steel space structure erected in Shenzhen City to some extent. The stress changes increase with increasing corrosion time, and they are larger in the case of double surface corrosion than in the case of inner surface corrosion only. It is worthwhile to point out that the results obtained from the structure concerned may not be applicable to other structures in other places. The framework proposed here can be used to perform assessment case by case. The observations made from the reticulated shell demonstrate that the changes in natural frequencies due to atmospheric corrosion are quite small. It is impossible to detect these small changes using modal tests because the small changes of natural frequency can be easily overlapped by noise contamination. Therefore, a reasonable way to monitor and assess the potential damage due to atmospheric corrosion is to install various sensors to monitor the climatic conditions and atmospheric contaminants. In addition, strain gauges are also needed to measure the stress states of important structural members which may be affected by the section loss caused by atmospheric corrosion. The monitoring of atmospheric corrosion damage of the large steel space structure and the reticulated shell will be discussed in detail in Chapter 4 and Chapter 9 respectively. The investigation of structural performance under the interaction of different corrosion damages, such as atmospheric corrosion and stress corrosion cracking, is important and will be discussed in the next chapter.
Table 3.1 Environmental parameters and index $N$

<table>
<thead>
<tr>
<th>Item / Site</th>
<th>Shenzhen</th>
<th>Beijing</th>
<th>Qingdao</th>
<th>Wuhan</th>
<th>Jiangjin</th>
<th>Guangzhou</th>
<th>Wanning</th>
<th>Qionghai</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_1$ Cl Deposition ($mg/100cm^2.d$)</td>
<td>0.037</td>
<td>0.049</td>
<td>0.250</td>
<td>0.011</td>
<td>0.006</td>
<td>0.024</td>
<td>0.387</td>
<td>0.199</td>
</tr>
<tr>
<td>$x_2$ SO$_2$ Deposition ($mg/100cm^2.d$)</td>
<td>0.118</td>
<td>0.442</td>
<td>0.704</td>
<td>0.272</td>
<td>0.667</td>
<td>0.107</td>
<td>0.060</td>
<td>0.150</td>
</tr>
<tr>
<td>$x_3$ NO$_2$ Content ($mg/m^3$)</td>
<td>0.043</td>
<td>0.220</td>
<td>0.038</td>
<td>0.089</td>
<td>0.007</td>
<td>0.035</td>
<td>0.005</td>
<td>0.008</td>
</tr>
<tr>
<td>$x_4$ Average temperature (°C)</td>
<td>22.1</td>
<td>11.9</td>
<td>12.3</td>
<td>16.8</td>
<td>17.9</td>
<td>22.9</td>
<td>24.2</td>
<td>24.3</td>
</tr>
<tr>
<td>$x_5$ Hours for relative humidity&gt;80% (h/a)</td>
<td>4730</td>
<td>2558</td>
<td>4049</td>
<td>4181</td>
<td>5741</td>
<td>4700</td>
<td>6020</td>
<td>6241</td>
</tr>
<tr>
<td>$x_6$ Precipitation (mm/a)</td>
<td>1908</td>
<td>586</td>
<td>562</td>
<td>1140</td>
<td>1203</td>
<td>1563</td>
<td>1515</td>
<td>1794</td>
</tr>
<tr>
<td>$x_7$ Precipitation days (days/a)</td>
<td>144</td>
<td>78</td>
<td>94</td>
<td>116</td>
<td>134</td>
<td>170</td>
<td>124</td>
<td>151</td>
</tr>
<tr>
<td>$x_8$ pH</td>
<td>4.7</td>
<td>5.5</td>
<td>6.1</td>
<td>6.5</td>
<td>4.4</td>
<td>5.8</td>
<td>5.0</td>
<td>6.9</td>
</tr>
<tr>
<td>$x_9$ Sunshine hours (h/a)</td>
<td>1822</td>
<td>2559</td>
<td>2161</td>
<td>1621</td>
<td>1371</td>
<td>1607</td>
<td>2026</td>
<td>2072</td>
</tr>
<tr>
<td>N Index</td>
<td>1.113</td>
<td>1.607</td>
<td>2.001</td>
<td>1.438</td>
<td>1.790</td>
<td>1.115</td>
<td>1.483</td>
<td>1.459</td>
</tr>
</tbody>
</table>

Table 3.2 Corrosion development trend $n$ for different types of metal material in China (Hou et al., 1994)

<table>
<thead>
<tr>
<th>Type</th>
<th>Beijing</th>
<th>Qingdao</th>
<th>Wuhan</th>
<th>Jiangjin</th>
<th>Guangzhou</th>
<th>Wanning</th>
<th>Qionghai</th>
</tr>
</thead>
<tbody>
<tr>
<td>D36</td>
<td>0.37</td>
<td>0.55</td>
<td>0.31</td>
<td>0.46</td>
<td>0.51</td>
<td>0.64</td>
<td>0.90</td>
</tr>
<tr>
<td>16MnQ</td>
<td>0.36</td>
<td>0.51</td>
<td>0.43</td>
<td>0.49</td>
<td>0.64</td>
<td>0.59</td>
<td>0.93</td>
</tr>
<tr>
<td>12CrMnCu</td>
<td>0.33</td>
<td>0.38</td>
<td>0.36</td>
<td>0.52</td>
<td>0.55</td>
<td>0.76</td>
<td>0.73</td>
</tr>
<tr>
<td>3C</td>
<td>0.35</td>
<td>0.51</td>
<td>0.60</td>
<td>0.47</td>
<td>0.49</td>
<td>1.06</td>
<td>0.66</td>
</tr>
<tr>
<td>09MnNb</td>
<td>0.50</td>
<td>0.49</td>
<td>0.41</td>
<td>0.43</td>
<td>0.53</td>
<td>1.04</td>
<td>0.88</td>
</tr>
<tr>
<td>A3</td>
<td>0.35</td>
<td>0.44</td>
<td>0.42</td>
<td>0.45</td>
<td>0.45</td>
<td>1.10</td>
<td>0.66</td>
</tr>
<tr>
<td>20</td>
<td>0.27</td>
<td>0.52</td>
<td>0.30</td>
<td>0.47</td>
<td>0.47</td>
<td>1.14</td>
<td>0.74</td>
</tr>
<tr>
<td>16Mn</td>
<td>0.36</td>
<td>0.54</td>
<td>0.56</td>
<td>0.42</td>
<td>0.48</td>
<td>1.37</td>
<td>0.94</td>
</tr>
<tr>
<td>08Al</td>
<td>0.42</td>
<td>0.56</td>
<td>0.69</td>
<td>0.59</td>
<td>0.61</td>
<td>1.97</td>
<td>1.50</td>
</tr>
</tbody>
</table>

3-38
Table 3.3 Coefficients $C_1$ and $C_2$ for different types of metal material in China (Wang et al., 1995)

<table>
<thead>
<tr>
<th>Material</th>
<th>$C_1$</th>
<th>$C_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>D36</td>
<td>34.85</td>
<td>50.16</td>
</tr>
<tr>
<td>16MnQ</td>
<td>33.88</td>
<td>51.88</td>
</tr>
<tr>
<td>12CrMnCu</td>
<td>36.24</td>
<td>51.40</td>
</tr>
<tr>
<td>3C</td>
<td>35.86</td>
<td>52.00</td>
</tr>
<tr>
<td>09MnNb</td>
<td>35.11</td>
<td>68.95</td>
</tr>
<tr>
<td>A3</td>
<td>39.36</td>
<td>61.34</td>
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<tr>
<td>20</td>
<td>42.09</td>
<td>74.97</td>
</tr>
<tr>
<td>16Mn</td>
<td>39.55</td>
<td>65.94</td>
</tr>
<tr>
<td>08Al</td>
<td>44.60</td>
<td>176.55</td>
</tr>
</tbody>
</table>

Table 3.4 Stress changes in the two structural members of the highest stress level due to inner surface corrosion

<table>
<thead>
<tr>
<th>Element</th>
<th>No corrosion</th>
<th>4</th>
<th>10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>8911</td>
<td>Stress level (%)</td>
<td>78.3</td>
<td>79.3</td>
<td>79.8</td>
<td>80.3</td>
</tr>
<tr>
<td>Stress change (%)</td>
<td>0</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
<td>3.1</td>
</tr>
<tr>
<td>9412</td>
<td>Stress level (%)</td>
<td>70.0</td>
<td>70.7</td>
<td>71.1</td>
<td>71.5</td>
</tr>
<tr>
<td>Stress change (%)</td>
<td>0</td>
<td>0.7</td>
<td>1.1</td>
<td>1.5</td>
<td>2.3</td>
</tr>
</tbody>
</table>
Table 3.5 The first 50 frequencies of the reticulated shell (Hz)

<table>
<thead>
<tr>
<th>No.</th>
<th>1</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.066</td>
<td>4.541</td>
<td>4.818</td>
<td>5.111</td>
<td>5.285</td>
</tr>
<tr>
<td>2</td>
<td>4.066</td>
<td>4.541</td>
<td>4.875</td>
<td>5.189</td>
<td>5.303</td>
</tr>
<tr>
<td>3</td>
<td>4.406</td>
<td>4.561</td>
<td>4.953</td>
<td>5.189</td>
<td>5.303</td>
</tr>
<tr>
<td>4</td>
<td>4.406</td>
<td>4.594</td>
<td>4.953</td>
<td>5.226</td>
<td>5.332</td>
</tr>
<tr>
<td>5</td>
<td>4.421</td>
<td>4.617</td>
<td>4.987</td>
<td>5.227</td>
<td>5.332</td>
</tr>
<tr>
<td>6</td>
<td>4.421</td>
<td>4.695</td>
<td>4.987</td>
<td>5.269</td>
<td>5.378</td>
</tr>
<tr>
<td>7</td>
<td>4.451</td>
<td>4.695</td>
<td>4.998</td>
<td>5.270</td>
<td>5.4372</td>
</tr>
<tr>
<td>8</td>
<td>4.452</td>
<td>4.708</td>
<td>5.069</td>
<td>5.271</td>
<td>5.440</td>
</tr>
<tr>
<td>9</td>
<td>4.483</td>
<td>4.726</td>
<td>5.069</td>
<td>5.271</td>
<td>5.440</td>
</tr>
<tr>
<td>10</td>
<td>4.483</td>
<td>4.818</td>
<td>5.109</td>
<td>5.276</td>
<td>5.465</td>
</tr>
</tbody>
</table>
Figure 3.1 Cross sections of typical structural members used in steel space structure:
(a) Circular section; (b) Rectangular section.
Figure 3.2 Procedure of nonlinear static analysis for steel space structure subjected to atmospheric corrosion.
Figure 3.3 Front view of the steel space structure in Shenzhen (2005).

Figure 3.4 Bird’s eye view of the steel space structure in Shenzhen (2005).
Figure 3.5 Elevation of middle part structure.
Figure 3.6 Plane view of middle part structure.
Figure 3.7 Structural components: (a) Top layer (sxg); (b) Top brace system (sfg); (c) Middle layer (zxg); (d) Bottom brace system (xfg); (e) Bottom layer (xxg); (f) Truss girders (hjt); (g) Tree-shaped supports (zcg).
Figure 3.8 The first 10 natural frequencies and mode shapes of the structure.
Figure 3.9 Ratio of working stress to yield stress of all the structural members.

Figure 3.10 Location of Shenzhen and other 7 cities in China.
Figure 3.11 Matching degree of Shenzhen to other 7 cities.

Figure 3.12 Variation of corrosion depth of materials with time.

Figure 3.13 Variation of corrosion rate of materials with time.
Figure 3.14 Variation of corrosion depth with SO₂ deposition.

Figure 3.15 Variation of corrosion depth with Cl⁻ deposition.

Figure 3.16 Variation of corrosion depth with NO₂ deposition.

Figure 3.17 Variation of corrosion depth with humidity hours.

Figure 3.18 Variation of corrosion depth with annual average temperature.

Figure 3.19 Variation of corrosion depth with sunshine hours per year.
Figure 3.20 Sensitivity of first 10 natural frequencies to thickness change of all the structural members (inner surface corrosion).
Figure 3.21 Frequency changes for different corrosion periods.

Figure 3.22 Variation of the first 5 natural frequencies with time.
Stress change under 10 years corrosion (%)

Stress change under 20 years corrosion (%)

Stress change under 50 years corrosion (%)

(a)
Figure 3.23 Statistics of structural members with various levels of stress change:
(a) Corrosion on inner surface; (b) Corrosion on both surfaces.
Figure 3.24 Stress changes in eight tree-shaped supports:
(a) Corrosion on inner surface; (b) Corrosion on both surfaces.
Figure 3.25 External view of the Astrodome during erection.

Figure 3.26 External view of Makomanai shell over the Sapporo Olympic Arena, Japan.
Figure 3.27 External view of some reticulated shells:
(a) City of London Exhibition Pavilion, England; (b) the Baton Rouge geodesic stressed-skin steel shell, USA; (c) Nagoya International Exhibition Centre, Japan; (d) Moka reticulated shell, Japan; (e) Olympic Badminton Gymnasium, China; (f) Olympic Bicycle Gymnasium, China.
Figure 3.28 Nodal and element number of the reticulated shell:
(a) Number of nodes; (b) Number of elements.
Figure 3.29 Static deformation of the reticulated shell: (a) 3D view of deformation; (b) Comparison of nodal deformation in various directions.
Figure 3.30 Internal forces of the reticulated shell:
(a) Axial force ($kN$); (b) Shear force $S_y$ ($kN$); (c) Shear force $S_z$ ($kN$); (d) Moment $M_x$ ($kN.m$); (e) Moment $M_y$ ($kN.m$); (f) Moment $M_z$ ($kN.m$).
Figure 3.31 Element stress and deformation of the reticulated shell:
(a) Maximum stress (MPa); (b) Maximum axial stress (MPa); (c) Maximum stress in circular members (MPa); (d) Maximum stress in radial members (MPa); (e) Maximum stress in skew members (MPa); (f) Axial deformation (cm).
Figure 3.32 Comparison of structural responses under two boundary conditions: (a) nodal displacement; (b) element axial stress.

Figure 3.33 Variation of the first 50 natural frequencies with span.

Figure 3.34 Variation of the first 50 natural frequencies with the $f/L$ ratio.
Figure 3.35 Variation of the first 50 natural frequencies with boundary conditions.

Figure 3.36 Matching degree of Shijiazhuang to other 7 cities.

Figure 3.37 Variation of corrosion depth of materials with time.

Figure 3.38 Variation of corrosion rate of materials with time.
Figure 3.39 Sensitivity of first 8 natural frequencies to thickness of all the structural members (inner and outer surface corrosion).
Figure 3.40 Frequency changes for different corrosion periods:
(a) Inner surface corrosion; (b) Double surface corrosion.

Figure 3.41 Variation of the first 5 natural frequencies with time:
(a) Inner surface corrosion; (b) Double surface corrosion.
Figure 3.42 Variation of maximum stress of members (MPa):
(a) 10 years corrosion; (b) 20 years corrosion; (c) 50 years corrosion.
CHAPTER 4

EVALUATION OF STRESS CORROSION CRACKING
OF STEEL SPACE STRUCTURES

4.1 INTRODUCTION

The atmospheric corrosion is a normal phenomenon resulting from the chemical or electrochemical action between humid environmental and metal material exposed in the open air (Roberge 2000; Revie 2000). A framework for the evaluation of potential damage due to atmospheric corrosion in steel space structures has been established and applied to a large steel space structure and a reticulated shell in Chapter 3 respectively. As mentioned in Chapter 3, the atmospheric corrosion occurs when the structure serves in the corrosion atmosphere and the occurrence of this corrosion process does not require the contribution of the mechanical stress of the member. The atmospheric corrosion is not a direct cause for the fracture or failure of the structural members. It may cause the section reduction of structural members which may lead to the stress redistribution in the structure to some extent. Apart from atmospheric corrosion, steel space structures exposed in the open air may suffer another kind of corrosion damage, stress corrosion cracking (SCC). SCC is a phenomenon in which propagation of member crack occurs under combined interaction of mechanical stress and corrosive environment (Russell 1992). Unlike the atmospheric corrosion, SCC may cause the crack propagation and even member fracture. Both mechanical stress and corrosive environment are essential for the occurrence of SCC. The stress required to cause SCC is commonly small and static tensile in nature. Structures serving in corrosive environment subjected to static loads
inevitably have the likelihood of SCC. If the accumulated damage caused by SCC cannot be timely detected, the structural safety will be threatened and the damage may finally cause the partial collapse of the structure, resulting in economic loss and fatal casualty (Revie 2000).

SCC is widely reported and studied in the chemical industry on the design and application of boilers and pressure vessels (Roberge 2000). Corresponding design codes and fabrication specifications have been provided by American Society of Mechanical Engineers (ASME 1997). The stress intensity factor (SIF) in fracture mechanics is commonly utilized to evaluate the feasibility of SCC for various types of boilers and pressure vessels (Barsom and Rolfe 1999). Similar to the industrial facilities, with the understanding on corrosion phenomenon and advance of stress corrosion research, the detection and evaluation techniques for stress corrosion damages of civil engineering structures have been developed. For civil engineering structures, the corrosion detection and evaluation of concrete structures and marine structures is the focus of attention due to their frequently reported corrosion damage. However, very little work has been carried out to evaluate the stress corrosion damage of steel structures. Fan et al (1994) performed the evaluation of load bearing capacity of an actual steel roof subjected to SCC in China. Sarveswarana et al (1998) proposed a method for investigating the reliability of corrosion-damaged steel members using interval probability theory and the data obtained from actual samples of corroded beams. Nevertheless, the current researches on stress corrosion damage of steel structures mainly focus on the corrosion damage evaluation of single structural components rather than the whole system. Unlike the industrial facilities, the service lives of
steel structures are commonly several decades and even more than one hundred years which are much longer than those of boilers and pressure vessels. The corrosive environment of steel space structures may vary significantly during this long service period and the variation of structural performance due to atmospheric corrosion such as section reduction and stress redistribution may accumulate to some extent. The variation of structural performance due to atmospheric corrosion will inevitably affect the extent of SCC. Therefore, the evaluation and monitoring approaches of steel space structures under the interaction of atmospheric corrosion and SCC require investigation.

In this regard, corrosion evaluation and monitoring approaches for steel space structures are investigated in this chapter through the interaction of atmospheric corrosion and SCC. The section reduction and stress redistribution due to atmospheric corrosion analyzed in Chapter 3 is utilized in the SCC evaluation of steel space structures. To simulate the possible stress corrosion damage, two types of cracks are adopted in the analysis. One is the semi-elliptical crack suggested by the ASME adopted to simulate the normal crack state. The other is the circumferential crack adopted to simulate the dangerous crack state. Then, the SIF in fracture mechanics is used to evaluate SCC of structure members with two selected types of cracks under the interaction of atmospheric corrosion. The large steel space structure built in southern coastal area of China adopted in Chapter 3 is also taken as an example to investigate damage characteristics of coupled atmospheric corrosion and SCC. The corresponding corrosion monitoring strategy and system for the steel space structure are presented. To be consistent, the issue of SCC in the reticulated shell is also discussed throughout the entire thesis.
4.2 **STRESS CORROSION DAMAGE**

As mentioned in Chapter 3, atmospheric corrosion shall cause the section loss of structural members of a steel space structure, which will in turn change the structural stiffness and lead to stress redistribution. The stress redistribution includes stress changes in both magnitude and direction for each member. In considering the fact that the stress inducing SCC is relatively small and tensile in nature, the stress redistribution due to atmospheric corrosion may affect the SCC of the structure. Thus, the coupled atmospheric corrosion and SCC damage evaluation is necessary for steel space structures. A framework for the evaluation of potential damage due to atmospheric corrosion to steel space structures has been established and applied to a large steel space structure in Chapter 3. The analytical procedure for determining the section loss and stress redistribution of steel space structure due to atmospheric corrosion is illustrated in detail. The member stress including the effects of atmospheric corrosion will be then utilized in the evaluation of SCC in this chapter.

Following the principle of fracture mechanics (Barsom and Rolfe 1999), the magnitude of the stress distribution at the crack tip which is the mechanical driving force for crack propagation is quantified by the SIF. Based on Griffith's original analysis of glass components with cracks and the subsequent extension of that work to ductile materials, the magnitude of the SIF is related directly to the magnitude of the applied stress and the square root of the crack size (Tada et al 1985). The crack SIF of member subjected to atmospheric corrosion is expressed as

\[
K = Y \sigma \sqrt{\pi a}
\]  

(4.1)

where \(a\) is the crack depth; \(Y\) is the correction factor which mainly depends on the geometry of a particular member and the geometry of crack. The correction factor \(Y\)
reflects the influence of many factors on SCC such as boundary condition, distance between different cracks, crack direction, and crack shape. \( \sigma_c \) is the uniform tensile stress of structural member induced by the structural static loads subjected to atmospheric corrosion. The calculating procedure of \( \sigma_c \) has been illustrated in Chapter 3 without considering crack.

Failures of engineering members may initiate at the member body or connection joint such as weld toe on the member surface. Commonly, the surface crack is the most common crack type observed on members of engineering structures. Practically, the cracks may have various types of shapes due to the diversity of member configuration and corrosion damages. For simplicity, ASME suggests that the irregular crack can be outlined as a regular semi-elliptical surface crack (See Figure 4.1) to simulate the common crack state in the absence of field inspection information (Gurney 1979; Barsom and Rolfe 1999). In Figure 4.1, \( t \) is the thickness of member wall; \( c \) is the crack width. For important civil engineering structures served in long-term periods in harsh environment, some serious damages are commonly assumed to occur in design stage to examine the functions of structures subjected to extreme events. The structure is therefore adequately designed to provide necessary safety provisions against catastrophic failure. The similar process is also adopted in the design of corrosion monitoring system for large steel space structure to fight against the coupled atmospheric corrosion and SCC in harsh corrosive environment after a long-term service. In this regard and in addition, the extremely dangerous circumferential crack configuration on the member surface is assumed to occur at the structural components due to various intensive mechanical and corrosive actions. The risk of coupled atmospheric corrosion and SCC of
structural members with circumferential cracks is accordingly evaluated based on SIF under structural dead loads.

For real steel space structures, the initial cracks may exist on the main body of structural member due to fabrication error or transportation damage, while the initial crack at connection joint is caused mainly by mechanical assemblage or welding process. The stress distribution around connection joint is different because of the stress concentration in the vicinity of the joint due to joint geometric discontinuity, the residual stress and defect at toe of the weld (Gurney 1979). To this end, the feasibility of SCC for both member body and connection joint is analyzed in this chapter. The computational procedure for the SIF of semi-elliptical crack and circumferential crack is presented. The risk evaluation of coupled atmospheric corrosion and SCC is carried out to demonstrate the properties of corrosion damage and explore the dangerous regions. Based on the observations, the corrosion monitoring system for steel space structures is established for structural safety assessment.

4.3 STRESS INTENSITY FACTOR

4.3.1 SIF for surface semi-elliptical crack

Owing to the difficulty in satisfying free boundary conditions, the accurate theoretical resolution to SIF of semi-elliptical surface crack is impossible at present, and only approximate result is available (Barsom and Rolfe 1999). The semi-elliptical crack on surface can be taken as the incision of elliptical crack in an infinite
body and the crack surface will be a free surface. The value of SIF for semi-elliptical surface crack of structural member under atmospheric corrosion can be written as

$$K = \frac{M_s M_p M_k}{\phi} \sigma^c \sqrt{\pi a}$$  \hspace{1cm} (4.2)$$

where $\phi$ is the integral factor and it can be determined by the following elliptic integral (Tada 1985)

$$\phi = \int_0^{\pi/2} \sqrt{1-k^2 \sin^2 \theta} \cdot d\theta$$ \hspace{1cm} (4.3)$$

$$k^2 = 1 - \frac{a^2}{c^2}$$ \hspace{1cm} (4.4)$$

The factor $M_p$ is the correction to allow for crack tip plasticity; the factor $M_s$ is the free-surface-correction factor whose value is about 1.12; $M_k$ is the back free-surface correction factor. To take the effective crack expansion into consideration, the SIF of member subjected to atmospheric corrosion is given by (Barsom 1999)

$$K = 1.12 \sigma^c \sqrt{\frac{\pi a}{Q}} \cdot M_k$$ \hspace{1cm} (4.5)$$

in which crack-shaper parameter $Q$ depends on $\sigma^c / \sigma_{ys}$ and $a / c$

$$Q = f\left(\phi, \frac{\sigma^c}{\sigma_{ys}}\right)$$ \hspace{1cm} (4.6)$$

where $\sigma_{ys}$ is the yield strength of the steel material. The value of $Q$ has been computed by many researches which is provided in reference book (Barsom 1999) for utilization. The factor $M_k$ is approximately 1.0 as long as the crack depth, $a$, is less than one-half the wall thickness, $t$. As $a$ approaches $t$, $M_k$ approaches approximately 1.6, and a useful approximation is provided by Barsom et al (1999)

$$M_k = 1.0 + 1.2 \left(\frac{a}{t} - 0.5\right) \quad \frac{a}{t} \geq 0.5$$ \hspace{1cm} (4.7)$$
It can be found that the stress intensity factor $K$ for semi-elliptical crack relates to crack dimension $a$ and $c$, uniform tensile stress $\sigma^e$ of structural member under atmospheric corrosion and yield strength of steel material $\sigma_{ys}$.

4.3.2 SIF for circumferential crack

The dangerous circumferential crack for typical rectangular and circular members used in steel space structures is plotted in Figure 4.2. Based on the work of Erdogan (1982) and Tada et al (1985), the SIF of rectangular member with circumferential is provided as

$$K = \sigma^e \sqrt{\pi a} \cdot F_r \left( \frac{a}{t} \right) \tag{4.8}$$

The shape coefficient $F_r(a/t)$ in Equation (4.8) is studied by many scholars such as Gross (1964), Emery (1969), Yamamoto (1972), and Tada (1985). The ratio of length to thickness $h/t$ will affect the values of $F_r$ while the effect of $h/t$ is practically negligible for $h/t > 1.0$. The expression for $F_r$ in Equation (4.8) can be expressed in the following equation with difference smaller than 0.5% for any $a/t$ (Tada, 1985)

$$F_r \left( \frac{a}{t} \right) = \sqrt{\frac{2t}{\pi a}} \tan \frac{\pi a}{2t} \cdot \frac{0.752 + 2.02 \left( \frac{a}{t} \right) + 0.37 \left( 1 - \sin \frac{\pi a}{2t} \right)}{\cos \frac{\pi a}{2t}} \tag{4.9}$$

The SIF of circular section for circumferential crack is studied relating to both $a/t$ and the ratio of inner and outer radius

$$K = \sigma^e \sqrt{\pi a} \cdot F_c \left( \frac{r_i}{r_o} \frac{a}{t} \right) \tag{4.10}$$

in which $r_i$ is the inner radius of the circular member; $r_o$ is the outer radius of the circular member; $a$ is the depth of the crack; $t$ is the thick of the member wall. The shape coefficient $F_c$ in Equation (4.10) is explored to be relative to the ratio of $r_i$ to $r_o$.
which does not have general expression for different $a/t$. Only numerical value of $F_c$ is provided in the SIF handbook (Tada 1985). For $r_i/r_o$ approaches 0 and $r_i/r_o$ approaches 1, Erdogan (1982) proved that the SIF of circular member can be expressed by omitting the contribution of $r_i/r_o$

$$K = \sigma \sqrt{\pi a} \cdot F_c \left( \frac{r_i}{r_o}, \frac{a}{t} \right) \approx \sigma \sqrt{\pi a} \cdot F_c \left( \frac{a}{t} \right)$$  \hspace{1cm} (4.11)

For $r_i/r_o$ approaches 0, the circular member behaves similarly to solid cylinder and the corresponding expression for shape coefficient $F_c$ is

$$F_c \left( \frac{a}{t} \right) = \frac{1.122 - 1.302 \cdot \left( \frac{a}{t} \right) + 0.988 \cdot \left( \frac{a}{t} \right)^2 - 0.308 \cdot \left( \frac{a}{t} \right)^3}{\left( 1 - \frac{a}{t} \right)^{3/2}}$$  \hspace{1cm} (4.12)

For $r_i/r_o$ approaches 1, the circular member behaves similarly to plate and the corresponding expression for shape coefficient $F_c$ is the same as that of the rectangular members which is displayed in Equation (4.9).

4.3.3 SIF for connection joint

For the real steel structure, the stress distribution at connection joint is quite complicated compared to that of member body. This is because the stress concentration is in the vicinity of the joint due to geometric discontinuities, the residual stress and defect at toe of the weld. The SIFs of crack near the connection joints are thereby different from those at the other areas to some extent. In this regard, the SIF $K_w$ at connection joint such as weld toe is approximately expressed as (Gurney 1979)

$$K_w = M_w K$$  \hspace{1cm} (4.13)
where $K$ is the SIF expressed in Equation (4.1) for member body; $M_w$ is the factor allowing for the fact that the crack is situated at a position of stress concentration at the weld toe due to geometric discontinuities and residual stress.

The relationship between $M_w$ and $a/t$ ratio can be numerically investigated through finite element analysis (Gurney et al 1976, 1978) and the value of $M_w$ has been found to depend on the leg length $l$ of the weld. A close approximation to the value of $M_w$ is given by (Gurney et al 1978)

$$
M_w = 0.8479 \left( \frac{l}{t} \right)^{0.063} \cdot \left( \frac{a}{t} \right)^{-0.279}
$$

(4.14)

For the transverse butt welds with the overfill in the shape of a circular arc, the corresponding value of $M_w$ is

$$
M_w = \left( \frac{2.5a}{t} \right)^{-q}
$$

(4.15)

$$
q = \frac{\log(11.584 - 0.0588\theta_w)}{\log(200)}
$$

(4.16)

where $\theta_w$ is the obtuse toe angle measured in degrees which is over the range of $135^\circ \leq \theta_w \leq 180^\circ$.

4.4 EVALUATION OF COUPLED ATMOSPHERIC CORROSION AND SCC

SCC is a phenomenon in which crack growth occurs when the necessary mechanical and corrosive conditions exist. The time duration from quick crack propagation to member fracture is short. This makes the SCC a brittle fracture process for structural members (Russell 1992). The researches up to now have revealed that no crack
propagation is observed if the stress level is below a certain threshold stress intensity level, $K_{ISCC}$, and the member fracture can be avoided

$$K < K_{ISCC} \quad (4.17)$$

This threshold stress intensity level $K_{ISCC}$ relates not only to the steel material but also to the corrosive environment around crack location of steel components. The value of $K_{ISCC}$ is commonly determined through experiment under the same corrosive environment and metal material to the actual ones (Revie 2000).

For the steel space structure, the SIF of member with cracks is computed based on the tensile stress under dead loads and corrosive environment and further compared with threshold SIF $K_{ISCC}$ to evaluate its likelihood of SCC. The SIF value substantially depends on the shape and size of crack which are commonly irregular and even unknown before inspection. In this regard, American Society of Mechanical Engineers (ASME) proposed a reference crack having a depth $a$ of 1/4 thickness $t$ and a length $c$ of 2 times depth $a$ for steel components (Hedden 2000) which is applied in the SCC design of pressure vessels and steel space structures (Fan and Xue 1994). The corrosion monitoring system is designed aiming to evaluate the risk of corrosion damage under cracks. In reality, not all the cracks can be effectively detected using the current nondestructive testing (NDT) approaches (Raj et al 2000). Practically, the minimum crack size that can be detected by the current NDT techniques is limited. For instance, the minimum crack size can be detected using well-adopted ultrasonic testing at present in China is about 1mm (Li and Liu 1999). Thus, the finer crack with width smaller than the allowable size in NDT techniques cannot be detected effectively. This kind of crack, however, still holds potential threats to the member safety under corrosive environment and deserves attention in
the monitoring process of corrosion damages. To this end, the crack size of 1mm for the minimum testing resolution of current ultrasonic testing approach in China is also assumed to occur at both the member body and connection joint to provide essential understanding on structural performance under undetected crack level.

For the monitoring and prevention of SCC in structures, different levels of acceptance should be developed to evaluate the damage extent and provide alarms for the owner to take essential measurements before the fracture events (Hedden 2000). The safety factor \( n_b \) is introduced by the ASME to make provision for the sudden fracture which is widely accepted in the prevention of SCC for industrial facilities

\[
K < \frac{K_{SCC}}{n_b} \tag{4.18}
\]

This criterion is also accepted in this chapter in the damage evaluation subjected to coupled atmospheric corrosion and SCC of steel space structures. The value of \( n_b \) is suggested as 2 for static strength design by ASME (ASME 1997, 1998). This value has been utilized in the SCC analysis of steel space structure in China (Fan and Xue 1994) and is also adopted in this chapter. The flow chart for the damage evaluation of coupled atmospheric corrosion and SCC for steel space structures is shown in Figure 4.3.

4.5 CORROSION DAMAGE EVALUATION OF EXAMPLE STRUCTURE

The SCC analysis of the steel space structure is carried out in this section to examine the structural safety under corrosive environment and dead loads. The stress redistribution of the example structure subjected to 50-year atmospheric corrosion is
computed in Chapter 3 through a nonlinear analysis based on Newton-Raphson iteration method. The obtained member stresses are utilized in the analysis of SCC. The SIFs of the structural members under tensile stress are calculated for semi-elliptical crack and circumferential crack respectively. The results are then compared with the threshold stress intensity level $K_{ISC}$ for the examination of the risk of SCC. As mentioned above, the threshold stress intensity level relates to both material and corrosive environment around crack, whose value is determined through experiment under the same corrosive environment and material type to the actual structural regions. For the initial analysis on SCC and design of corrosion monitoring system, no in-situ information of corrosive environment around structural members are available to carry out the experiment and determine $K_{ISC}$. In this regard, the value of $K_{ISC}$ obtained from the other experiment and field measurement is adopted in the analysis. The measurement on the value of $K_{ISC}$ for the steel space structure in service environment is very limited. According to the SCC evaluation of a steel space structure in China (Fan and Xue 1994), the threshold stress intensity level $K_{ISC}$ of construction steel material for steel space structures is 36.41MN/m$^{3/2}$, which is taken as the threshold value in this study for theoretical analysis.

4.5.1 SCC evaluation on member body

Based on the stress redistribution obtained in Chapter 3, the SIFs of member body for semi-elliptical and circumferential cracks under 50-year atmospheric corrosion are computed and plotted in Figure 4.4. Because the mechanical stress for SCC is tensile in nature, the SIFs of members in compressive stress are set at zero. The results in Figures 4.4 (a) and (b) demonstrate that the member SIFs under semi-elliptical
surface cracks are far smaller than the threshold stress intensity level $K_{ISCSC}$. While the member SIFs under circumferential cracks are much larger compared with those under semi-elliptical cracks as shown in Figures 4.4 (c) and (d). The SIF in Figure 4.4 (b) indicates that although it is still within the safe range, the SIFs of truss members (hjt) are much greater than those of other members with $0.25t$ crack depth. Similar conclusions can be drawn in the analysis of structural members with circumferential crack under 50 years atmospheric corrosion. Comparing SIFs between two types of cracks, it can be found that the potential hazard induced by circumferential crack is greater than that by surface semi-elliptical crack. Under only 1mm depth circumferential crack, the SIFs of a few members are close to warning level for crack expansion. Although member SIFs under $0.25t$ circumferential crack are lower than the threshold value, a few members in major truss (hjt) already exceed the warning level.

4.5.2 SCC evaluation on connection joint

As mentioned before, the SIF at connection joint is different from that at member body. This is because the stress concentration is in the vicinity of the joint due to geometric discontinuities, the residual stress and defect at toe of the weld. For simplicity, a factor $M_w$ is adopted by Gurney (1979) to approximately estimate the influence of geometric discontinuities and residual stress at weld toe on the SIF at connection joint. Based on this factor, the uniform tensile stress utilized in the computation can be selected as that of member body. The evaluation of coupled atmospheric corrosion and SCC for connection joint with crack is conducted in this section by adopting semi-elliptical and circumferential cracks. The SIF of connection joint adopting two kinds of cracks subjected to 50 years atmospheric corrosion are
numerically investigated and shown in Figure 4.5 for the members with tensile stresses. As shown in Figures 4.5 (a) and (b), all the connection joints do not have the risk of SCC with surface semi-elliptical cracks even under 50 years atmospheric corrosion. However, the SIFs under circumferential cracks are larger than those under semi-elliptical cracks as shown in Figures 4.5 (c) and (d) especially for truss members (hjt). Figure 4.5 (c) demonstrates that the SCC risk of some joints under 1mm circumferential is very close to the warning level which should deserve enough attention because this crack depth is too small to be accurately detected by the currently-adopted ultrasonic testing techniques in China. Figure 4.5 (d) clearly reveals that SIFs of some joints connected with truss members (hjt) exceed the warning level and have a relatively large risk of SCC under 0.25t circumferential cracks. The comparison of SIFs between member body and connection joint demonstrates that the SCC risk of connection joint is larger than that of member body to some extent for both semi-elliptical and circumferential cracks. The comparison between two assumed crack types reveals that the fracture risk under dangerous circumferential crack is substantially larger than that under surface semi-elliptical crack. These observations demonstrate that the structural members and connection joints do not have the risk of SCC under current dead loads. Essential monitoring techniques and field measurement works shall be taken to timely monitor the developing trend of cracks and prevent the occurrence of potential member damages such as circumferential cracks because these damage events may significantly increase the risk of member failure.
4.5.3 Effects of atmospheric corrosion on SCC

As described previously, long-term atmospheric corrosion would cause structural stiffness deterioration and stress redistribution, which will further induce the variation of SIF for member SCC. The variation of SIFs with time duration of atmospheric corrosion is also examined to understand the effects of atmospheric corrosion on SCC. Some connection joints with large SIFs are selected to examine the effects of atmospheric corrosion on SCC because these joints have larger risk of fracture than other joints. Three joints with large SIFs are selected from the top brace system (sfg, connected to member No.8911), top layer (sxg, connected to member No.11175) and truss girders (hjt, connected to member No.11968) respectively to investigate their changes in SIFs with corrosion time under circumferential cracks. The curves in Figure 4.6 manifest the distinction among different connection joints. As shown in Figure 4.6, the joint SIF of member 8911 reduces with time and joint SIF of member 1175 almost keep constant while the joint SIF of member 11968 is adversely changed. In addition, the varying amplitudes of SIF for different joints are different to some extent. The variation of joint SIF of member 8911 and 11968 is relatively large while that of member 11175 is quite small. Further investigation on the other connection joints reveals that the SIF variation of most joints with circumferential crack under 50 year atmospheric corrosion is in the range of 5% to 10%. Fortunately, for the example steel space structure, the increment of SIF due to section loss and stress redistribution induced by atmospheric corrosion is still not beyond the safe level. It is worthwhile to point out that this observation made from the concerned structure is obtained based on the current corrosive environment and working stresses. However, such results may not be applicable to other structures in other places.
4.6 DESIGN OF CORROSION MONITORING SYSTEM

This thesis aims to develop an integrated health monitoring and vibration control system for the reticulated shell. The design of the integrated system is conducted by including the effects of various loads and extreme events such as dead loads, wind, earthquake, instability, temperature changes, fire and corrosion. The corrosion monitoring is one of the tasks of the developed integrated monitoring and control system. The framework of developing a corrosion monitoring system is demonstrated in this section by taking the large steel space structure constructed in the Shenzhen as an example. This procedure can also be applied in the corrosion monitoring of the reticulated shell. The issue of corrosion monitoring of the reticulated shell will be included in the design of the integrated health monitoring and vibration control system, which will be discussed in detail in Chapter 9.

4.6.1 Objectives

The evaluation of atmospheric corrosion damage and SCC of the example steel space structure has been carried out in Chapter 3 and this chapter respectively. The observation made from the steel space structure is based on the current corrosive environment and working stresses. Actually, the service life of important steel space structures is commonly several decades and even more than one hundred years. During this long period, the corrosive environment of the structures may vary to a great extent. Therefore, the analytical model shall be updated gradually following the varying environmental information and the evaluation process of coupled atmospheric corrosion and SCC should be carried out regularly. To fulfill this task, long-term information of environmental information and structural stress distribution are necessary and therefore reasonable monitoring strategy and system should be
developed for the steel space structures. In this regard, a corrosion monitoring system is devised to monitor the potential damage due to combined interaction of atmospheric corrosion and SCC. This integrated system is conceived as a complicated system making use of the knowledge of structural analysis, corrosion science, and damage evaluation. The monitoring system based on the synthesis of structural analysis and corrosion knowledge with modem sensing and transmission techniques shall be established to direct and supervise the operation of the corrosion monitoring and evaluation of steel space structures. It is intended that when completely developed it will be a corrosion monitoring system used for the following purposes:

(1) To update the analytical model for corrosion damage and to improve the assessments of structural behavior, strength and durability of the steel space structure served in corrosive environment.

(2) To monitor the performance of the steel space structure in corrosive environment with the aims of assessing its safety, reducing failure risk and maintaining serviceability. This process will assist in defining priorities in inspection and maintenance of the entire structure.

(3) To develop knowledge of monitoring methods for large steel space structures which may advance the application of corrosion science, structural analysis and measurement technology to operation and maintenance of civil engineering structures and be used in the design and construction of other steel space structures in the future.
4.6.2 System design

Based on the above mentioned objectives, the design of the corrosion monitoring system for the example steel space structure is developed in this section. The analytical results obtained in Chapter 3 demonstrate that the changes in natural frequencies due to atmospheric corrosion are quite small. It is impossible to detect these small changes by measuring natural frequencies. The SCC risk of structural components can be evaluated by using the information of member stresses and crack configuration in corrosive environment. Therefore, a reasonable way to monitor and assess the potential corrosion damage is to install various sensors to monitor the corrosive environment and structural stress distribution. To develop reasonable monitoring strategy and system, the characteristics of atmospheric environment in which the structure serves is firstly described to understand the possible varying trends of corrosive environment. Then, the sensory system is designed based on the aforementioned analyses of atmospheric corrosion and SCC aiming to collect various environmental information and structural responses. The collected environmental information and structural response shall be stored in different databases and analyzed based on developed analytical model and evaluation procedure for safety assessment. Finally, the layout of the corrosion monitoring system is demonstrated.

4.6.2.1 Environmental conditions

As mentioned before, after a long-term service, the corrosive environment affecting the structure performance may vary to some extent. The analytical model and evaluation procedure for corrosion damage shall be updated gradually following the varying environmental information. To develop reasonable monitoring strategy and
system, possible varying trends of corrosive environment should be firstly understood.

The Shenzhen City is located in the southern coastal areas of China adjacent to Hong Kong. The areas of city region have enlarged from 315km² in 1980 to 2020km² now and simultaneously the population increases from 30 thousand in 1980 to about 6 million at present with the transformation from a rural area to a modern coastal metropolis. Shenzhen City presents the subtropical climate with the characteristics such as long summer, short winter, abundance of precipitation and sunshine (Zhong et al., 2001, 2002). This city is controlled by the corrosive marine and industrial atmosphere. Figure 4.7 displays the variation of some environmental parameters with time in Shenzhen based on the meteorological measurement results (Zhong et al., 2001, 2002). Figures 4.7 (a) and (b) display the variation of annual average ambient temperature and monthly temperature with time for the past 50 years. It can be learned from the figures that the average ambient temperature fluctuated before 1980s while the temperature rapidly increased since 1980 with the expansion of city scale and the increase in population. The monthly average ambient temperature is close to 20 degrees Celsius even in winter while the difference between maximum and minimum temperature is quite large for almost every month. The annual precipitation plotted in Figure 4.7 (c) demonstrates that the precipitation fluctuates around 2000mm to some extent and presents a very slight increasing trend after 1980. Figure 4.7 (d) indicates the distribution of monthly average precipitation and rainy days which clearly indicates that the most important rainy periods is from May to September. The annual sunshine hour in Shenzhen city reduces gradually as shown in Table 4.1 for different time periods. Similar to the observations from annual average temperature, there exists a distinct reduction of sunshine hour after 1980. It
is reported (Zhong et al 2001, 2002) that the reasons for descending sunshine hours are complicated. The air pollution (the increase of fog and clouds), human activity, and environmental variation (the increase of building structures, change of topography and vegetation) may affect the changes in sunshine hour. Figure 4.8 displays the change in NO₂ content in the air and rain PH value with time during the past several years. Clearly, the contaminants in the atmosphere increase and therefore the influence of corrosive rain on steel space structure may gradually increase. It is reported that in Shenzhen, the NO₂ in the atmosphere mainly comes from the automobile exhaust and the SO₂ ion and Cl⁻ mainly come from industrial exhaust gas and corrosive marine monsoon (Huang et al 2001). With the rapid development of industry and the increase in automobiles, the contents of corrosive contaminants in atmosphere gradually increase. The corrosive air contaminants shall be dissolved in the rain to form corrosive acid rain and accelerate the corrosion of steel space structures.

The statistical properties of the environmental conditions clearly demonstrate that the rapid increase in size and population of the city from 1980 substantially change the environmental parameters. The variation of these environmental parameters will affect the corrosion states of the steel members. Therefore, one can concluded that the atmospheric environment for the example steel space structure may gradually deteriorate which may accelerate the steel corrosion and increase the accumulated corrosion damages. Therefore, the evaluation of the coupled atmospheric corrosion damage and SCC of steel space structure should be carried out regularly based on the varying environmental parameters. In addition, the prediction model of atmospheric
corrosion should be updated based on the measured information of environmental parameters and material corrosion depth.

4.6.2.2 Sensor arrangement

The corrosion monitoring system is designed aiming to monitor the corrosive environment, estimate corrosion damage and evaluate safety of the large steel space structure after a long-term service. Based on the analyses in Chapter 3 and this chapter, it is understood that the occurrence of the atmospheric corrosion damage and SCC relates to the corrosive environment, stress level and crack configuration which thereby are the crucial aspects in the design of corrosion monitoring system. In this regard, the monitoring process is basically divided into three categories: environmental monitoring, member corrosion and crack monitoring, and structural response monitoring. The environmental monitoring refers to the monitoring of wind, temperature, humidity, sunshine, precipitation, rain PH value and atmospheric contaminants. The member corrosion and crack monitoring refers to the monitoring of corrosion states of steel members and inspection of various cracks. The structural response monitoring refers to the monitoring of the member stress.

The tree-shape supports and truss members connected with spherical bearings are the major supporting components of the entire structure. These parts have relatively complicated configuration and can easily accumulate rain and moisture than other parts of the frame as shown in Figure 4.9. This moist environment easily leads to the coupled atmospheric corrosion and SCC. In addition, the results of sensitivity analysis discussed in Chapter 3 reveal that the tree-shape supports and truss members are the most sensitive components to atmospheric corrosion. Failures of these two
supporting components would inevitably cause the collapse of the entire structure. Therefore, the tree-shape supports and truss members are the crucial parts for the installation of sensors. Figures 4.10 (a) and (b) display the SIFs for connection joints of tree-shape supports with semi-elliptical cracks and circumferential cracks respectively. Despite the different crack configuration, the distribution of SIF values among various groups is similar and the components of tree-shape supports with large SIF for SCC are groups 1, 4, 5 and 8, which are candidate position for the installation of sensors. Similarly, the SIF distribution for connection joints of truss members is plotted in Figures 4.11 (a) and (b) for semi-elliptical cracks and circumferential cracks respectively. It can be seen that the members with relatively large SIF values are close to the spherical bearings of both circular tower and rectangular tower. Actually as mentioned above, these areas can easy accumulate rain and moisture (See Figure 4.9), therefore the corrosion depth of steel components in these areas should be closely monitored.

The type, amount and distribution of sensors should be determined to achieve the goals of monitoring climate variation, atmospheric contaminants, material corrosion and structural stress distribution. The monitoring system for the concerned large steel space structure includes many kinds of sensors: acidimeter, anemometer, hygrometer, pyranometer, ombrometer, temperature sensor, SO\textsubscript{2} sensor, NO\textsubscript{2} sensor, Cl\textsuperscript{−} sensor, corrosion sensor and strain gauge. The configuration of the sensory system for the steel space structure is plotted in Figure 4.12 to demonstrate the type, amount, and distribution of various sensors excluding strain gauge (which will be discussed later).
Because the content of the atmospheric contaminants directly relates to the velocity and direction of air flows, an anemometer is installed at the top of the circular tower to record the in-situ wind information and further explore the variation trends of atmospheric contaminants with wind. An ombrometer and an acidimeter are required to monitor the precipitation, precipitation days and the rain PH value. Moreover, a pyranometer is installed to record the variation of sunshine hours. The anemometer, ombrometer, acidimeter and pyranometer are installed at the top of concrete towers to avoid the influence of two concrete towers which are about 13m higher than the top of frame roof. As discussed above, the tree-shape support group 1 is an important supporting system of the entire structure and is sensitive to atmospheric corrosion damage. Therefore, a group of contaminant sensors (SO₂ sensor, NO₂ sensor and Cl⁻ sensor) is placed to monitor the atmospheric contaminants. Similarly, a group of contaminant sensors is also required to monitor the atmospheric contaminants near the members around spherical bearings. Considering the member SIFs plotted in Figure 4.11 and the spatial distribution of two groups of contaminant sensors in the entire structure, it is decided to install the other group of contaminant sensors near the spherical bearing No. 25 adjacent to the rectangular tower.

As discussed above, the truss members close to spherical bearings have relatively large SIF values. Therefore, these members have larger risk of SCC than other members. In addition, the regions around spherical bearings can easy accumulate rain and moisture leading to corrosion damage events. Therefore, twelve corrosion sensors are placed near the spherical bearings of both circular tower and rectangular tower with one close to one spherical bearing as shown in Figure 4.11. Similarly, four corrosion sensors are installed on tree-shape supports of groups 1, 4, 5 and 8
respectively because these members have large SIF values. In addition, other six corrosion sensors are distributed on the edges and corners of the space frame to measure the possible material corrosion as shown in Figure 4.11. The corrosion depth of steel component measured by the corrosion sensors can be utilized to update the analytical model for coupled atmospheric corrosion and SCC. In addition, the chemical constitution of the corrosion media in these areas can be also measured and simulated in the laboratory to determine the threshold stress intensity level $K_{ISC}$. Because the material corrosion and member stress will be substantially affected by ambient temperature, it is thus decided to place ambient temperature sensors with the same number and distribution as those of corrosion sensors. Furthermore, four hygrometers are installed on the tree-shapes supports and spherical bearings for monitoring the humidity of crucial parts easily subjected to corrosion damage. The number and distribution of sensors are listed in Table 4.2.

The strain gauge in monitoring system can be placed with relatively large amount due to its low cost. The analytical results of SCC under 0.25 $t$ circumferential cracks are taken as the basis for distribution of strain gauge and periodic field inspection on cracks. As mentioned before, different levels of acceptance should be developed to evaluate the corrosion damage extent and provide alarms for the owner to take essential measurements before the failure events. The safety factor $n_p$ is selected as 2 to provide the threshold for alarm of potential SCC. In this chapter, the adopted threshold value of SIF for stress monitoring and crack measurement is lower than that for safety alarm to provide more safety provisions against corrosion damage. In this regard, the threshold value of SIF for determining the number and location of strain gauges and crack measurement is computed by taking safety factor $n_p$ as 3.
The number and distribution of strain gauges in corrosion monitoring for both member body and connection joint are computed under 0.25t circumferential cracks and listed in Table 4.3. Clearly, the most dangerous components for the coupled atmospheric corrosion and SCC of the steel space structure are the truss members close to concrete towers.

4.6.2.3 Databases

The corrosion monitoring system aims to monitor the environment variation, structural corrosion damage, and evaluate the safety of the example steel space structure after a long-term service. The system consists of many kinds of sensors: acidimeter, anemometer, hygrometer, pyranometer, ombrometer, ambient temperature sensor, SO₂ sensor, NO₂ sensor, Cl⁻ sensor, corrosion sensor and strain gauge. The data collected from sensory system and field inspection should be analyzed based on developed analytical model and evaluation procedure. Before the safety assessment, the collected information should be stored in different databases for further analysis. Therefore, five databases are designed for the corrosion monitoring system in line with different data sources: database for climate information; database for atmospheric contaminant information; database for stress responses; database for material corrosion information; and database for member crack information. The envisaged tasks in the formation and processing of these five databases are described as follows:

(1) Database for climate information collects the information of wind, ambient temperature, humidity, sunshine, precipitation, and rain PH value. This information will be utilized in the prediction of the section loss of structural members due to
atmospheric corrosion. Furthermore, statistical analysis shall be carried out to reveal developing trends of these parameters and the relationship among them.

(2) Database for air contaminant information collects the measured information of contaminants such as NO₂, SO₂ and Cl⁻. Similar to climate information, air contaminant information shall be utilized in the prediction of the atmospheric corrosion damage of the steel space structure. The information of climatic parameters and air contaminant will be utilized in the updating of prediction model for atmospheric corrosion. In addition, the air contaminants shall be dissolved in the accumulated rain and moisture to form the corrosive environment which may produce the SCC. Therefore, the chemical constitution of this corrosion media shall be regularly measured and simulated in the laboratory to determine the threshold stress intensity level $K_{SCC}$. Moreover, the evaluation procedure of SCC shall be modulated based on the experimental value of $K_{SCC}$. Furthermore, if the contents of atmospheric contaminants vary substantially during short time periods, it is reasonably to increase the frequency of regular measurement on chemical constitution of corrosion media and corresponding laboratory test is required to modify the evaluation procedure of SCC.

(3) Database for stress responses collects the stress information of member body and connection joint estimated based on measurement and theoretical analysis. Because only responses of limited members can be obtained, the analytical model of the structure will be updated based on the limited information. Then, the stresses of those members without being monitored by sensors could be predicted. The information of tensile stress will be combined with the information of member crack for the evaluation of coupled atmospheric corrosion and SCC.
(4) Database for material corrosion information collects the corrosion information of structural components such as member body and connection joint. The measured information of section loss under atmospheric corrosion will be combined with the monitoring results of climatic factors and contaminants to update the prediction model for atmospheric corrosion.

(5) Database for member crack information records the number, distribution and configuration of various cracks through periodic field inspection and measurement. The information of the crack configuration will be utilized to update the evaluation procedure of SCC.

4.6.2.4 System layout

The evaluation of potential damages due to atmospheric corrosion in steel space structures has been carried out in Chapter 3. In addition, the SCC evaluation is also conducted in this chapter for the examination of the potential damages due to stress corrosion. Corresponding parameter investigation is conducted for both atmospheric corrosion and SCC to explore the effects of environmental parameters and crack configuration on structural performance and safety. These observations can be referred to as the design guidance for the corrosion monitoring system. Based on above analyses, the conceptual system layout and operation of the corrosion monitoring system for the example steel space structure are developed and displayed in Figure 4.13. The flow chart in Figure 4.13 demonstrates that the system configuration based on structural analysis, corrosion knowledge and measurement technology with the establishment of five databases for corrosion damage assessment. The corrosion monitoring system of the steel space structure obtains the stress responses under dead loads through the combination of measurement information
and theoretical results. This information is therefore utilized in the evaluation of coupled atmospheric corrosion and SCC. The sensory system of the steel space structure consists of four categories: (1) the sensors used to monitor the climatic variation which includes acidimeter, anemometer, hygrometer, pyranometer, ombrometer and temperature sensor; (2) sensors used to monitor the contents of atmospheric contaminants which includes SO$_2$ sensor, NO$_2$ sensor and Cl$^-$ sensor; (3) strain gauges to record the strain of member body and connection joint; and (4) the corrosion sensors installed in the crucial parts to provide corrosion information of steel components.

Based on the obtained information from sensory system and field measurement, the information of climatic parameters, air contaminant, and material corrosion damage will be utilized in the updating of prediction model for atmospheric corrosion. In addition, the chemical constitution of corrosion media formed from the dissolution of air contaminants in accumulated rain and moisture shall be regularly measured and simulated in the laboratory to determine the threshold stress intensity level $K_{ISC C}$.

The evaluation procedure of SCC therefore shall be modulated based on the experimental value of $K_{ISC C}$. Furthermore, if the contents of atmospheric contaminants vary substantially during short time periods, it is reasonably to increase the frequency of regular measurement on chemical constitution of corrosion media and corresponding laboratory test is required to modify the evaluation procedure of SCC. After that, the system analysis and safety evaluation are carried out again to demonstrate the structural damages under corrosive environment which is provided as the guidelines of decision making for structural safety and maintenance.
4.7 SCC OF RETICULATED SHELL

Similar to the analytical procedure adopted for the large steel space structure, the evaluation of coupled atmospheric corrosion and SCC of the reticulated shell is carried out. The observations made in Chapter 3 demonstrate that the changes in natural frequencies due to atmospheric corrosion are quite small for the reticulated shell. Furthermore, the analytical results clearly demonstrate that all the members of the reticulated shell have compressive forces under current dead loads. The long-term atmospheric corrosion may cause slight stress redistribution which shall affect slightly the magnitude of the member stress. This, however, shall not lead to a change from the compressive stress to tensile stress. Therefore, the reticulated shell in this study does not have the risk of SCC. It should be pointed out that this observation of the reticulated shell may not be applicable to other reticulated shells in other places. The evaluation procedure of coupled atmospheric corrosion and SCC developed in this chapter can be used to perform assessment case by case.

4.8 SUMMARY

The procedure of developing a framework for evaluating and monitoring corrosion damage of steel space structures by incorporating knowledge of atmospheric corrosion, SCC, and structural analysis has been proposed in this chapter. The evaluation method for atmospheric corrosion damage is illustrated through the modified prediction model of atmospheric corrosion, sensitivity technique and nonlinear static analysis procedure in Chapter 3. The analytical procedure of coupled atmospheric corrosion and SCC is presented based on the SIF expression in fracture mechanics to evaluate the SCC under two types of cracks. The SCC of connection
joint is evaluated by utilizing modification factors to involve the effects of stress concentration in the vicinity of joint. The large steel space structure adopted in Chapter 3 is taken as the example to examine the feasibility and effectiveness of the proposed framework to evaluate and monitor possible corrosion damage. The case study on SCC is conducted to explore the features of corrosion damage as well as crucial components which are provided as the guidelines for the design of corrosion monitoring system. The effects of atmospheric corrosion on SCC are taken into consideration to examine the interaction of these two types of corrosion damages. Numerical investigation demonstrates that the stress corrosion damage of the large steel space structure subjected to dead loads will not cause the fracture of structural components while several components of truss members close to concrete towers exceed the warning level of SCC. The circumferential crack is more dangerous than the semi-elliptical crack at both member body and connection joint. The measured environmental and atmospheric information reveals that the rapid expansion of city scale and population substantially affect the city climate and increase the contaminants in atmosphere which may increase the corrosion damages of the steel space structure.

The monitoring system for corrosion damage is conceived as an environment-based system involving the techniques of structural analysis, damage evaluation and field inspection to monitor the structural performance in corrosive environment and develop knowledge of monitoring strategy of steel space structures. This may advance the application of corrosion knowledge, structural analysis and measurement technology to operation and maintenance of civil engineering structures. Five databases are constructed to record the information of climate, atmospheric
contaminants, stress responses, material corrosion and cracks. The sensor arrangement for the large steel space structure is illustrated to achieve the goals of monitoring climate variation, atmospheric contaminants, material corrosion and member stress and evaluating corrosion damage. The potential risk of coupled atmospheric corrosion and SCC is also discussed in this chapter to assess the structural safety. The design of corrosion monitoring system for the reticulated shell is not included in this chapter. This will be carried out in Chapter 9 which discusses the design of the integrated health monitoring and vibration control system for the reticulated shell. The framework presented in this chapter can also be utilized for corrosion damage assessment and monitoring of other steel space structures in the future.
Table 4.1 Annual sunshine hours in different time periods

<table>
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<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Average sunshine hour</td>
<td>2267</td>
<td>2112</td>
<td>1884</td>
<td>1882</td>
</tr>
</tbody>
</table>

Table 4.2 Number and distribution of sensors

<table>
<thead>
<tr>
<th>Number</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anemometer</td>
<td>1</td>
</tr>
<tr>
<td>Acidimeter</td>
<td>1</td>
</tr>
<tr>
<td>Ombrometer</td>
<td>1</td>
</tr>
<tr>
<td>Pyranometer</td>
<td>1</td>
</tr>
<tr>
<td>Hygrometer</td>
<td>4</td>
</tr>
<tr>
<td>Temperature sensor</td>
<td>22</td>
</tr>
<tr>
<td>Corrosion sensor</td>
<td>22</td>
</tr>
<tr>
<td>SO₂ sensor</td>
<td>2</td>
</tr>
<tr>
<td>NO₂ sensor</td>
<td>2</td>
</tr>
<tr>
<td>Cl⁻ sensor</td>
<td>2</td>
</tr>
</tbody>
</table>

Table 4.3 Number and distribution of strain gauges

<table>
<thead>
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<th></th>
<th>zcg</th>
<th>xxg</th>
<th>xfg</th>
<th>zyg</th>
<th>sfg</th>
<th>sxg</th>
<th>hjt</th>
<th>total</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>3</td>
<td>3</td>
<td>23</td>
<td>29</td>
</tr>
<tr>
<td>Connection joint</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>3</td>
<td>42</td>
<td>49</td>
</tr>
</tbody>
</table>
Figure 4.1 Steel member containing a semi-elliptical surface crack.

Figure 4.2 Circumferential crack profiles of typical structural members used in steel space structure: (a) Circular section; (b) Rectangular section.
Figure 4.3 Flow chart for the evaluation of SCC of the steel space structure under atmospheric corrosion.
Figure 4.4 SIFs of member body under 50 years atmospheric corrosion.
Figure 4.5 SIFs of connection joints under 50 years atmospheric corrosion.
Figure 4.6 Variation of joint SIF with time duration of atmospheric corrosion.
Figure 4.7 Variation of environmental factors with time for Shenzhen city: (a) Annual average temperature; (b) Monthly temperature; (c) Annual average precipitation; (d) Monthly precipitation and rainy days.

Figure 4.8 Variation of NO₂ content and rain PH value with time.
Figure 4.9 Pictures of supports of the steel space structure:
(a) Tree-shaped supports; (b) Supports on circular tower.
Figure 4.10 SIFs of connection joints of tree-shaped supports (zcg):

(a) Semi-elliptical crack; (b) Circumferential crack.
Figure 4.11 SIFs of connection joints of the truss girders (hjt):
(a) Semi-elliptical crack; (b) Circumferential crack.
Figure 4.12 Sensor configuration for monitoring of climate and atmosphere contaminants: (a) Plan view; (b) The tree-shaped supports; (c) Elevation view.
Figure 4.13 Layout of corrosion monitoring system for steel space structure.

- Measurement of environment, corrosion information and structural responses
  - Climate monitoring
  - Atmospheric pollution monitoring
  - Strain monitoring

- Database for climate information
- Database for atmospheric information
- Database for strain responses
- Database for material corrosion information
- Database for structural crack information

- Extents of corrosion damage
- Evaluation of atmospheric corrosion damage
- Evaluation of SCC under atmospheric corrosion

- System analysis and safety evaluation
- Member and joint cracks (NDT)

- Display
- Decision making
- Routine monitoring
- Serviceability
- Recording
- Further inspection
- Alarm
- Further inspection

- Yes: Update evaluation model of corrosion damage?
- No

Legend:
- ACM - Acidimeters
- ANM - Anemometers
- HYM - Hygrometers
- PYM - Pyranometers
- OMM - Ombrometers
- TES - Temperature Sensors
- SO2 S - SO2 sensors
- Cl- S - Cl- sensors
- NO2 S - NO2 sensors
- SG - Strain gauges
- CS - Corrosion sensors
- NDT - Non-destructive Testing
- SCC - Stress corrosion cracking
CHAPTER 5

DAMAGE DETECTION DUE TO SUDDEN STIFFNESS CHANGE USING INSTANTANEOUS INDEX

5.1 INTRODUCTION

Steel space structures may undergo corrosion damage if they are exposed in the corrosive environment after a long-term service. A framework for the evaluation of potential damage due to atmospheric corrosion in steel space structures is proposed in Chapter 3. The assessment of coupled atmospheric corrosion and stress corrosion cracking (SCC) is made in Chapter 4 to provide an integrated procedure for the evaluation techniques and monitoring approach on corrosion damages of steel space structures. The SCC shall lead to the member fracture which may cause sudden stiffness changes in structures. Although the numerical investigation in Chapter 3 demonstrates that all the members of the reticulated shell work in compressive states under dead loads. Therefore, the reticulated shell does not have the risk of sudden fracture due to SCC under dead loads. Besides the SCC, the structural instability under external loads may cause buckling of structural components which may further lead to sudden stiffness reduction. In reality, the instability of civil engineering structures has been widely reported (Beles et al., 1967; Smith et al., 1980) in the past decades. Therefore, it is necessary to develop an effective approach to detect the damage event caused by possible sudden stiffness reduction.
The sudden reduction of structural stiffness will cause a discontinuity in acceleration response time histories recorded in the vicinity of damage location at damage time instant. Hou et al (1999, 2000) firstly proposed a wavelet transform (WT) approach to identify the damage time instant and damage location of a simple structural model with breakage springs. The same idea for detecting sudden damage was adopted by Vincent et al (1999) and Yang et al (2001) but using empirical mode decomposition (EMD), developed by Huang et al (1998, 1999), to decompose the vibration signal to capture the signal discontinuity. Numerical simulation carried out in their studies showed that the EMD approach could also identify the damage time instant and damage location using the signal feature of damage spike. In addition to the above-mentioned numerical studies, Xu and Chen (2004) carried out experimental studies on the applicability of EMD for detecting structural damage caused by a sudden change in structural stiffness. A series of free vibration, random vibration, and earthquake simulation tests were performed on a three-storey shear building model with different damage time instants and damage severities simulated. Their experimental results confirmed that the EMD approach with intermittency check could accurately identify the damage time instant and damage location. However, both the numerical study and the experimental investigation demonstrate that the relationship between damage spike amplitude and damage severity could not be given by the WT and the EMD with intermittency check. To this end, Yang et al (2004) suggested an alternative method based on the EMD with intermittency check and Hilbert transform (HT) to quantitatively detect the damage time instant and the natural frequencies and damping ratios of the structure before and after damage. However, this multi-stage method proposed by Yang et al (2004) may not be suitable for online structural health monitoring applications because it needs great
computation effort. Therefore, it is necessary to develop an automatic and quick detection approach for the sudden damage event for the online health monitoring process.

In this chapter, detection procedure for the sudden stiffness change using the EMD with intermittency check is briefly introduced. After that, the signal discontinuities in the acceleration response time histories recorded in the vicinity of damage location due to a sudden damage event are closely examined for both single-degree-of-freedom (SDOF) systems and MDOF systems. An instantaneous damage index is then proposed to detect the damage time instant, location, and severity of a structure due to a sudden change in stiffness. The proposed damage index is proportional to damage severity and suitable for online structural health monitoring applications. It can also be used in conjunction with EMD for damage detection without using the intermittency check. Numerical simulation using a five-storey shear building as an example is carried out to assess the effectiveness and reliability of the proposed damage detection approach for different types of excitations and at different levels of damage. The sensitivity of the damage index to the intensity and frequency range of measurement noise is also examined to provide further understanding on detection performance. Based on the performance examination on the shear building, the proposed damage detection approaches are applied to the reticulated shell. The stability analysis of the shell is thus conducted first to explore the regions suffered sudden damage event which will be utilized in the design of damage scenario for the reticulated shell. The detection approaches are then used to detect the time instant and location of sudden damage events of the shell. The effects of noise contamination and damage severity on detection performance are finally examined.
The damage detection results from the proposed approaches are also compared with those from WT approach. The installation principle of accelerometers for monitoring sudden damage event is also discussed. The arrangement of accelerometers in the reticulated shell will be carried out in Chapter 9 which discusses the design of the integrated health monitoring and vibration control system for the reticulated shell.

5.2 **Empirical mode decomposition**

As a new signal processing method, the EMD can decompose any data set into several intrinsic mode functions (IMFs) by a procedure called sifting process (Huang et al. 1998, 1999). An IMF is defined as a function that satisfies two conditions: (1) Within the data range, the number of extrema and the number of zero crossings are equal or differ at most by one; and (2) at any point, the mean value of the envelope defined by the local maxima and the envelope defined by the local minima is zero.

Suppose $X(t)$ is a time history (signal) to be decomposed. The sifting process is conducted by first constructing the upper and lower envelopes of $X(t)$ by connecting its local maxima and local minima through a cubic spline. Designate the mean value of the two envelopes as $m_1(t)$ and compute the difference between the original time history and the mean value of the two envelopes as $h_1(t)$ and compute the difference between the original time history and the mean value

$$h_1(t) = X(t) - m_1(t)$$

(5.1)

The component $h_1(t)$ is then examined to see if it satisfies the above-mentioned two requirements to be an IMF. If not, the sifting process is to be repeated by treating $h_1(t)$ as a new time history until $h_1(t)$ is an IMF, designated as $c_1(t)$. Then, the first IMF is separated from the original time history, giving a residue $r_1(t)$ as

$$r_1(t) = X(t) - c_1(t)$$

(5.2)
The sifting process is applied successively to each subsequent residue to obtain the subsequent IMFs until either the residue \( r_n(t) \) is smaller than a predetermined value or it becomes a monotonic function. The original time history is finally expressed as the sum of the IMF components plus the final residue.

\[
X(t) = \sum_{j=1}^{n} c_j(t) + r_n(t)
\]  

(5.3)

where \( c_j(t) \) is the \( j \)th IMF component; \( n \) is the total number of IMF components; and \( r_n(t) \) is the final residue. After the decomposition, the first IMF component obtained has the highest frequency content of the original time history while the final residue represents the component of the lowest frequency in the time history. During the sifting process, a criterion called intermittency check (Huang et al., 1999) can be imposed for each IMF component in order to limit its frequency content. This criterion works by specifying a frequency, termed intermittency frequency \( f_c \), for each IMF component during its sifting process, so that the data having frequencies lower than \( f_c \) will be removed from the resulting IMF.

As illustrated by Hou et al (1999, 2000), the damage signal has the characteristics of higher frequency. Therefore, the first IMF obtained by adopting intermittency check is utilized to detect the time instant and location of sudden damage event (Vincent et al., 1999; Yang et al., 2001). To illustrate the detection process of sudden stiffness reduction using the EMD with intermittency check, a SDOF system is numerically investigated here. The mass and stiffness of the SDOF system examined are

\[
m = 1.117 \times 10^4 \text{ kg} \quad \text{and} \quad k = 1.75 \times 10^5 \text{ N/m}.
\]

The damping ratio of the system is set as 2%. The SDOF system is subjected to El Centro 1940 earthquake ground excitation with a peak amplitude 1.0m/s². The system is supposed to suffer a sudden 20%...
stiffness reduction at \( t=4.0 \text{s} \). The acceleration response time history is computed using the Newmark-\( \beta \) method with a time step of 0.002s. The two factors in the Newmark-\( \beta \) method are selected as \( \alpha = 1/2 \) and \( \beta = 1/4 \).

The acceleration response time history and its first three IMFs are plotted in Figure 5.1. The first IMF is obtained using intermittency check (\( f_c = 125 \text{Hz} \)). It is seen that the direct inspection on acceleration response cannot explore the damage events. As shown in Figure 5.1 (b), a spike exists in the first IMF with highest frequency content to demonstrate the time instant of sudden stiffness reduction. The similar spike cannot be observed in the second and the third IMFs of the acceleration response (See Figures 5.1(b) and (c)). The detection approach using the EMD with intermittency check can provide other information on damage in the time-frequency domain as done by Yang et al (2004) in conjunction with the Hilbert transform. Since this approach needs a time history of a certain length to carry out EMD, small time delay is unavoidable in terms of online damage detection. Therefore, it is necessary to develop an automatic and quick detection approach for the sudden damage event for the online health monitoring system.

### 5.3 SIGNAL FEATURE DUE TO SUDDEN DAMAGE

#### 5.3.1 Signal feature due to sudden damage-SDOF system

Let us consider a single-degree-of-freedom (SDOF) system subjected to a sudden stiffness reduction under impulse excitation. The mass of the system is denoted as \( m \), the damping ratio \( \xi \) of the system is supposed to remain unchanged before and after
sudden damage, and the stiffness is denoted as \( k \) which will have a sudden reduction at time instant \( t_i \)

\[
k = \begin{cases} 
  k_u & (0 \leq t \leq t_i) \\
  k_d & (t_i < t)
\end{cases}
\]  

(5.4)

In which \( k_u \) and \( k_d \) are the stiffness of undamaged and damaged system, respectively.

The initial velocity and displacement due to the impulse excitation are assumed to be \( v_0 \) and 0, respectively. The circular frequency of the system before and after sudden damage can be express as

\[
\omega_u = \sqrt{\frac{k_u}{m}}; \quad \omega_d = \sqrt{\frac{k_d}{m}}
\]  

(5.5)

By defining a frequency reduction coefficient \( \alpha \) that varies from 0 to 1

\[
\omega_d = \alpha \cdot \omega_u \quad (0 < \alpha < 1)
\]  

(5.6)

The stiffness reduction can be expressed as

\[
\Delta k = k_d - k_u = m(\omega_d^2 - \omega_u^2) = ma_0^2(\alpha^2 - 1)
\]  

(5.7)

The equation of motion of the SDOF system before sudden damage is

\[
y'' + 2\zeta \omega_u y' + \omega_u^2 y = 0
\]  

(5.8)

The above equation can be solved in terms of the given initial conditions, and the structural responses at time instant \( t_i \) are

\[
y_i = e^{-\zeta \omega_u t_i} \cdot \left[ \frac{v_0 \sin(\omega_u \sqrt{1-\zeta^2} \cdot t_i)}{\omega_u \sqrt{1-\zeta^2}} \right]
\]

\[
y_i = e^{-\zeta \omega_u t_i} \cdot \left[ \frac{v_0 \cos(\omega_u \sqrt{1-\zeta^2} \cdot t_i) - \xi v_0 \sin(\omega_u \sqrt{1-\zeta^2} \cdot t_i)}{\sqrt{1-\zeta^2}} \right]
\]  

(5.9)

Let us take the time instant \( t_i \) as the start point of the SDOF system after sudden damage and use a new time axis \( t = t - t_i \). Then, the equation of motion of the system after damage becomes
\[
\ddot{y}_i + 2\xi\omega_u\dot{y}_i + \omega_u^2 y_i = 0 \tag{5.10}
\]

The initial conditions for Equation (5.10) can be expressed as

\[
y_i(0) = y_i = e^{-\xi\omega_u t_i} \cdot \frac{v_0 \sin(\omega_u \sqrt{1 - \xi^2} \cdot t_i)}{\omega_u \sqrt{1 - \xi^2}}
\]

\[
\dot{y}_i(0) = \dot{y}_i = e^{-\xi\omega_u t_i} \left[ v_0 \cos(\omega_u \sqrt{1 - \xi^2} \cdot t_i) - \frac{\xi v_0 \sin(\omega_u \sqrt{1 - \xi^2} \cdot t_i)}{\sqrt{1 - \xi^2}} \right] \tag{5.11}
\]

The damping ratio of a civil engineering structure is often very small, that is, \(1 - \xi^2 \approx 1\). Furthermore, the time interval \(\Delta t_i = t_{i+1} - t_i\) should be very small to describe the sudden stiffness reduction properly. As a result, one may have

\[
\omega_u \sqrt{1 - \xi^2} \cdot \Delta t_i \to 0 \Rightarrow \begin{cases} 
\cos(\omega_u \sqrt{1 - \xi^2} \cdot \Delta t_i) \to 1 \\
\sin(\omega_u \sqrt{1 - \xi^2} \cdot \Delta t_i) \to 0 \end{cases} \tag{5.12}
\]

The relationship between the acceleration response increment and the sudden stiffness reduction can thus be given approximately as

\[
\ddot{y}_{i+1} - \ddot{y}_i \approx -\frac{\Delta k}{m\omega_u} \cdot e^{-\xi\omega_u t_i} \cdot \sqrt{1 - \xi^2} \cdot v_0 \sin(\omega_u t_i \cdot \sqrt{1 - \xi^2}) \tag{5.13}
\]

Equation (5.13) reveals that the acceleration response increment is approximately linear to the sudden stiffness reduction for given initial velocity, damage instant and structural parameters before damage. If the time interval \(\Delta t_i\) for sudden damage is further regarded as a fixed value, Equation (5.13) indicates that the acceleration response discontinuity due to sudden stiffness reduction can be reflected by the slope of the acceleration response at damage instant.

To examine the slope of the acceleration response of the system at other time instant in addition to the damage time instant, a numerical example is discussed in the following based on the slope of acceleration response given by
\[ K_k = \frac{\dot{y}_{k+1} - \dot{y}_k}{\Delta t} \quad (k = 1, 2, ..., n-1) \]  

(5.14)

where \( \Delta t = t_{k+1} - t_k = \Delta t; \) and \( n \) is the total number of time interval. The mass and stiffness of the SDOF system examined are \( m = 1.117 \times 10^4 \text{kg} \) and \( k = 1.75 \times 10^5 \text{N/m} \). The damping ratio of the system is set as 2\%. The SDOF system is subjected to an impulse, represented by 0.1\text{m/s} initial velocity. The system is supposed to suffer a sudden 5\% stiffness reduction at \( t=2.3s \). The acceleration response time history is computed with a time interval of 0.002\text{s} and plotted in Figure 5.2(a), and the corresponding acceleration response slope time history is depicted in Figure 5.2 (b) using its absolute value. It can be seen that there is a spike at the damage time instant \( t_i \). The absolute values of acceleration response slope at the time instant \( t_{i-1} \) and the time instant \( t_{i+1} \) are much smaller than that at the damage time instant \( t_i \).

\[ |K_i| \gg |K_{i-1}|; \quad |K_i| \gg |K_{i+1}| \]  

(5.15)

The above two features cannot be seen at any other time instant without sudden stiffness reduction.

5.3.2 Signal feature due to sudden damage-MDOF system

To examine the signal feature due to a sudden stiffness reduction in a multi-degree-of-freedom (MDOF) system, the acceleration responses of a five-storey shear building, as shown in Figure 5.3, with a sudden change of stiffness at its first storey and subjected to various types of external excitations are investigated numerically. The mass and horizontal stiffness of the undamaged building are uniform for all storeys with a mass \( m = 1.3 \times 10^6 \text{kg} \) and a horizontal stiffness \( k = 4.0 \times 10^9 \text{N/m} \). The Rayleigh damping assumption is adopted to construct the structural damping matrix,
and the damping ratios in the first two modes of vibration of the building are set as 0.05. The original building is supposed to suffer a sudden 20% stiffness reduction in the first storey with the horizontal stiffness reducing from $4.0 \times 10^9$ N/m to $3.2 \times 10^9$ N/m while the horizontal stiffness in other storeys remains unchanged (no damage). Listed in Table 5.1 are the natural frequencies of the original and damaged building. The frequency reduction due to 20% stiffness reduction in the first storey is small with a maximum reduction no more than 5% in the first natural frequency. The frequency reduction becomes even smaller in higher natural frequencies. Three types of external excitations: seismic excitation, sinusoidal excitation, and impulse excitation, are respectively considered to compute the acceleration response of the building to have a wide collection of signal features due to sudden damage. The seismic excitation used is the El-Centro 1940 earthquake ground acceleration (S-N component) with a peak amplitude $1.0 \, m/s^2$. The sinusoidal excitation is expressed by Equation (5.16) and assumed to act on each floor of the building.

$$f(t) = 1300 \cdot \sin(4\pi t) \quad (0 \leq t \leq 10 \, s) \quad (kN)$$ (5.16)

The impulse excitation is supposed to occur at the first floor of the building only, represented by $0.1 \, m/s$ initial velocity of the first floor. The damage time instant of the building is set as $6.0 \, s$ for seismic excitation and sinusoidal excitation and $0.2 \, s$ for impulse excitation. The equation of motion of the building subjected to a sudden reduction of 20% horizontal stiffness at its first storey at the given time instant is then established for each type of external excitation, and it is solved using the Newmark-$\beta$ method with a time step of $0.002 \, s$. The two factors in the Newmark-$\beta$ method are selected as $\alpha = 1/2$ and $\beta = 1/4$. 5-10
The computed acceleration response of the first floor of the building is displayed in Figure 5.4 (a) for seismic excitation, in Figure 5.5 (a) for sinusoidal excitation, and in Figure 5.6 (a) for impulse excitation. It is difficult to find the signal feature due to sudden damage by direct visual inspection of the original acceleration responses. The 0.2 second portion of the acceleration response time history, containing the damage time instant, is expanded in Figure 5.4 (b) for seismic excitation and in Figure 5.5 (b) for sinusoidal excitation, and the 0.12 second portion of the same quantity is expanded in Figure 5.6 (b) for impulse excitation to permit a close look at signal feature due to sudden damage. It can be seen that there exists a sudden jump in the original signal at the damage time instant. This kind of signal jump cannot be observed at other time instants. The sudden reduction of horizontal stiffness of the first floor causes a clear signal discontinuity in the acceleration response time history at the damage time instant. Since the signal discontinuity is of very high frequency, the EMD is applied to decompose the original acceleration response without using intermittency check. As a result, eight, six, and six IMFs are obtained from the acceleration response time history at the first floor of the building under seismic excitation, sinusoidal excitation, and impulse excitation, respectively. Displayed in Figure 5.4 (c) to Figure 5.4 (e) are the first three IMFs of the acceleration response at the first floor of the building under seismic excitation. Similar to the original acceleration response time history, the direct visual inspection of the IMF components cannot find signal features due to sudden damage. The 0.2 second zooms of the three IMFs around the damage time instant are therefore depicted in Figure 5.4 (f) to Figure 5.4 (h). It can then be seen that the 0.2 second zoom of the first IMF is quite similar to the 0.2 second portion of the original acceleration response time history, and the signal discontinuity is reserved in the first IMF
component only. This is because the first IMF component often contains the highest frequency component of the original signal. The first three IMFs obtained from the original acceleration response time history at the first floor of the building under sinusoidal excitation and impulse excitation and their local zooms around the damage time instant are plotted in Figure 5.5 and Figure 5.6, respectively. Similar observations can be made, that is, the local zoom of the first IMF is quite similar to the local portion of the corresponding acceleration response time history and the signal discontinuity is reserved in the first IMF only.

To extract inherent signal feature due to sudden damage from the signal discontinuity in either the original acceleration response time history or the first IMF of the original time history, the acceleration responses of the building under each type of excitation are computed for a sudden reduction of stiffness at the first storey with different damage levels and damage time instants. Signal discontinuity patterns are then searched from extensive computed results, and all the possible patterns are plotted in Figure 5.7. It can be seen from Figure 5.7 that the signal discontinuity due to sudden stiffness change at the damage time instant \( t_i \) possesses two common distinct features: (1) the amplitude of acceleration signal \( \ddot{y} \) jumps up or down considerably from the damage time instant \( t_i \) to the time instant \( t_{i+1} \); and (2) the absolute values of signal slope at the time instant \( t_{i-1} \) and the time instant \( t_{i+1} \) are much smaller than that at the damage time instant \( t_i \). These two features are the same as those observed from the SDOF system.
5.3.3 Instantaneous damage index

In considering the aforementioned two signal discontinuity features and having an index more sensitive to sudden damage, a damage index, $DI_i$, is defined to reflect the signal discontinuity due to sudden damage at the time instant $t_i$:

$$DI_i = |(K_i - K_{i-1}) + (K_i - K_{i+1})| = |2K_i - K_{i-1} - K_{i+1}| \quad (i = 2, 3, ..., n - 1) \quad (5.17)$$

This damage index is computed in the time domain and it is an instantaneous index suitable for online structural health monitoring application. Furthermore, the damage indices, $DI_{i-1}$ and $DI_{i+1}$, at the time instants $t_{i-1}$ and $t_{i+1}$ can be calculated by

$$DI_{i-1} = |2K_{i-1} - K_{i-2} - K_i| \quad (5.18)$$

$$DI_{i+1} = |2K_{i+1} - K_{i} - K_{i+2}| \quad (5.19)$$

In consideration of Equation (5.17), the summation of the damage indices, $DI_{i-1}$ and $DI_{i+1}$, at the time instants $t_{i-1}$ and $t_{i+1}$ is approximately equal to the damage index $DI_i$ at the damage time instant $t_i$, that is,

$$DI_{i-1} + DI_{i+1} \approx DI_i \quad (5.20)$$

As a result, a common damage index pattern consists of one relatively large damage index $DI_i$ at the damage time instant $t_i$ and two relatively small indices $DI_{i-1}$ and $DI_{i+1}$, satisfying Equation (5.20), at the time instants $t_{i-1}$ and $t_{i+1}$ as shown in Figure 5.8. The damage index at any other time instant without sudden stiffness reduction should be very small. This damage index pattern can help exclude some false damage indices. Furthermore, the linear relationship between the proposed damage index and the sudden stiffness reduction still remains

$$DI_i = |2K_i - K_{i-1} - K_{i+1}| \approx |2K_i| \propto |K_i| \propto |\Delta k| \quad (5.21)$$
This damage index pattern can help exclude some false damage indices. Furthermore, the linear relationship between the proposed damage index and the sudden stiffness reduction still remains.

5.3.4 Two damage detection approaches

Based on the above discussions, two damage detection approaches can be proposed to detect sudden damage of a structure in the time domain. One is to analyze the original acceleration response time history directly to obtain the variation of damage index with time, from which the damage event can be identified. The other is to use the EMD without intermittency check to decompose the original acceleration response time history to obtain its first IMF component. The variation of damage index with time is then computed basing on the first IMF to determine the damage event. The general steps involved in each damage detection approach are shown in Figure 5.9. Clearly, the first approach can be implemented in an online health monitoring system without any time delay while the second approach needs a time history of certain length to carry out the EMD and accordingly small time delay is unavoidable in terms of online damage detection. Nevertheless, the second approach gains an additional insight into the feature of sudden damage. Moreover, in conjunction with the Hilbert transform the second approach can provide other information on damage in the time-frequency domain as done by Yang et al (2004).

5.4 DAMAGE DETECTION OF SHEAR BUILDING

5.4.1 Damage detection under various excitations
To examine the feasibility of the proposed damage index and damage detection approaches for identifying damage time instant and location, the acceleration responses of the aforementioned five-storey shear building to the seismic excitation are computed. The building is subjected to a 20% sudden stiffness reduction at time $t=6.0$ second in the first storey of the building only. The time step used in the computation is 0.002 second. The two approaches are then applied respectively to the acceleration response of each floor to calculate the damage index. Figure 5.10 (a) to Figure 5.10 (e) shows variation of damage index with time for each floor of the building under the seismic excitation, obtained by the first approach without using EMD. Figure 5.10 (f) to Figure 5.10 (j) display the same quantities but they are obtained by the second approach using the first IMF. It can be seen that no matter which approach is used, the damage index of the first floor is very large only at time $t=6.0$ second, which is exactly the moment when the stiffness of the first storey is suddenly reduced by 20%. The damage indices of the first floor at all other time instants (except for time $t=5.998$ second, 6.0 second, and 6.002 second) are very small so that the damage index at time $t=6.0$ second looks like a spike. Therefore, the damage time instant can be easily identified by the occurrence time of the sharp damage index. Let us now compare the variation of damage index of the first floor (Figure 5.10 (a)) with those of the second, third, forth, and fifth floors of the building (Figure 5.10 (b) to Figure 5.10 (e)). The sharp damage index appears clearly only in the first floor, and no sharp damage index emerges in other floors. Therefore, by analyzing the distribution of sharp damage index along the height of the building, the damage location can be easily identified at the first storey of the building. The same conclusion with respect to the damage location can be reached using the second approach with the first IMF as shown in Figure 5.10 (f) to Figure 5.10 (j).
The variation of damage index with time for the building is shown in Figure 5.11 for sinusoidal excitation and in Figure 5.12 for impulse excitation using the two approaches. The building is subject to the same damage severity at the first storey only but it occurs at time $t=6.0$ second for sinusoidal excitation and at time $t=0.2$ second for impulse excitation. Again, the sharp damage index appears only at the moment of sudden stiffness reduction at the first floor. Thus, the damage time instant and location can be easily captured from the observed occurrence time of the sharp damage index and its distribution along the height of the building. As expected, for the building under a given excitation the magnitude of the sharp damage index obtained by the first approach is almost the same as that obtained by the second approach. To check the relationship expressed by Equation (5.20), the damage indices computed around the damage time instant for each type of excitation are listed in Table 5.2. In Table 5.2, the item $\Delta DI_i$ is defined as follows:

$$\Delta DI_i = \frac{|DI_{i+1} + DI_{i+1} - DI_i|}{|DI_i|} \times 100\%$$  

(5.22)

Clearly, Equation (5.22) exists for the three excitation cases studied here and the damage index patterns are consistent with those presented in Figure 5.8. However, it can be observed that the magnitude of sharp damage index is different for the same building under different types of excitations. The magnitude of sharp damage index depends on many factors such as type of excitation, damage time instant and damage severity, which will be discussed subsequently.

To further examine the feasibility of the proposed damage index and damage detection approaches, parameter studies are carried out in this section to find the sensitivity of damage index to damage severity. The first floor of the five-storey
building is supposed to suffer different levels of sudden stiffness reduction but the sudden reduction occurs at the same time. Listed in Table 5.1 are the damage severities and the corresponding five natural frequencies of the building before and after the sudden damage. The results presented in Table 5.1 indicate that the stiffness reduction in the first storey of the building affects mainly on lower natural frequencies. It is noted that if the stiffness reduction in the first floor is less than 20%, the maximum frequency change is no more than 5%. Since the proposed two approaches give almost the same results, only the results from the second approach using the first IMF are presented and discussed in the following.

The 0.15 second portion of the first IMF of the acceleration response at the first floor of the building to the seismic excitation is plotted in Figure 5.13 for the sudden stiffness reduction from 1% to 40%. The variation of damage index with time obtained from the first IMF is also depicted in the same figure. It can be seen that even for small damage event such as 1% to 5% sudden stiffness reduction, the proposed approach can easily capture the damage features for noise free situation. The magnitude of sharp damage index also increases with increasing damage severity. Similar results are also obtained for the building subjected to sinusoidal excitation as shown in Figure 5.14. For the building under impulse excitation, however, the proposed approaches may not give satisfactory results for the building with small damage event (1% to 2% sudden stiffness reduction) as shown in Figure 5.15. This is because the signal fluctuates significantly immediately after the initial velocity. Nevertheless, if the damage index pattern shown in Figure 5.8 and expressed by Equation (5.20) is utilized with the following criterion
the damage detection results will be improved for the small damage events as shown in Figure 5.16. In Equation (5.23), \( T \) is the threshold value and it is 0.2 used to obtain the results in Figure 5.16.

### 5.4.2 Relationship between damage index and damage severity

The magnitude of sharp damage index corresponding to different damage severities in the first storey of the building subjected to seismic, sinusoidal and impulse excitations is computed and listed in Table 5.3 for the first approach without using EMD and in Table 5.4 for the second approach basing on the first IMF. They are also plotted in Figure 5.17 together with a linear fit in which \( x \) represents damage severity (stiffness reduction) and \( y \) represents damage index. It can be seen that the relationship between damage index and damage severity is quite linear for the building under either seismic or sinusoidal or impulse excitation. For a given external excitation, the larger the stiffness reduction, the larger is the damage index. Also for a given external excitation, the linear relationship obtained by the first approach without using EMD is very close to that obtained by the second approach basing on the first IMF. However, the slope of the linear fit is different for the building under different excitations. This creates a problem for the sole determination of damage severity. One possible way of solving this problem is to measure external excitation in addition to structural responses. The proposed approach is then used to find the damage time instant and damage location from the measured structural responses. Afterwards, the measured external excitation is input to the structural model with a sudden stiffness reduction at the identified damage location and at the identified
damage time instant to determine the slope of the linear relationship between the
damage severity and damage index. The linear relationship can finally used to
determine the damage severity in the actual structure in terms of the damage index
identified from the actual structure.

5.4.3 Effects of noise on detection under various excitations

To apply the proposed approaches to health monitoring and damage detection of real
structures, many practical issues need to be addressed. The effect of measurement
noise, on the detection of real structural damage is a key issue. Yang et al (2004)
reported that the damage spike identified from the first IMF component using the
intermittency check could be weaken by measurement noise, and strong
measurement noise could lead to the failure of damage detection. In consideration
that the sudden damage event introduces a high frequency component to acceleration
responses of a structure, the effects of both measurement noise intensity and
frequency range on the damage detection using the proposed approaches are
investigated in this study. The measurement noise in structural response is assumed
to be a random white noise. Three frequency ranges are considered: (1) white noise
with frequency range from 0 to 50Hz; (2) white noise with frequency range from 0 to
100Hz; and (3) white noise with frequency range from 0 to 250Hz. The measurement
noise intensity is defined as

\[
\text{Noise intensity} = \frac{\text{RMS}(\text{noise})}{\text{RMS}(\text{signal})} \times 100\% \tag{5.24}
\]

Displayed in Figure 5.18 are the 0.15 second zooms of the first IMFs of the
contaminated acceleration responses of the five-storey building at the first floor
under the seismic excitation for two noise intensities and three noise frequency
ranges. The sudden stiffness reduction in the first storey of the building is 20%. It
can be seen that with increasing noise frequency range, the acceleration response of
the first floor becomes more fluctuating and the signal discontinuity at the damage
time instant gets weak. The variation of the corresponding damage index obtained by
using the second approach with the first IMF is provided in Figure 5.19 for the
seismic excitation case. Clearly, the proposed approach can still identify the damage
time instant from the contaminated acceleration response of the building at the first
floor for the designated two noise intensities and three noise frequency ranges.
Furthermore, by checking the spatial distribution of sharp damage index along the
height of the building, the proposed approach can also identify the damage location
from the contaminated acceleration response for the designated two noise intensities
and three noise frequency ranges. The same results are obtained for the sinusoidal
excitation case, as shown in Figure 5.20. However, for the impulse excitation case,
the proposed approach fails to identify the damage time instant and damage location
when the noise frequency range is from 0 to 250 Hz and the noise intensity is 5%, as
shown in Figure 5.21.

The effects of measurement noise on the magnitude of damage index are assessed
and the results are listed in Table 5.5 for seismic excitation, in Table 5.6 for
sinusoidal excitation, and in Table 5.7 for impulse excitation. It can be seen that as
long as the damage event can be identified, the magnitude of damage index remains
almost the same for the designated two noise levels and three noise frequency ranges
no matter which approach is used. This indicates that the effect of measurement
noise on the magnitude of damage index is small. This is probably because the
damage index defined here is an instantaneous damage index. The further numerical
simulation indicates that the damage can be effectively identified for 20% sudden
stiffness reduction even with 80% noise intensity as long as the noise frequency range is not higher than 50 Hz. However, if the noise frequency range is from 0 to 250 Hz, the reliability of damage detection using the proposed approaches significantly deteriorates with the increase in noise intensity.

It should be pointed out that the shear building is assumed to have a sudden change of stiffness only at its first storey in the above investigation. The results from the further numerical investigation demonstrate that the damage events at other floors can also be effectively detected without/with noise contamination. It is observed that if the sudden stiffness reduction occurs at the Nth floor of the building (N > 1), the signal discontinuity can be observed only in the acceleration responses of the Nth and the (N-1)th floor. Therefore, it is easy to determine the damage location based on the spatial distribution of damage index. The linear relationships between damage severity and damage index can be observed in the acceleration responses of both the Nth and the (N-1)th floors. These results are not displayed here for the sake of conciseness.

5.5 Damage Detection of Reticulated Shell

As mentioned before, SCC of steel space structures leads to the likelihood of crack propagation and fracture of structural components which shall cause sudden stiffness reduction. For the reticulated shell investigated in this study, all the members are in compressive states and do not have the risk of SCC under dead loads. In reality, the shell may undergo instability if it is subjected to strong external loads after a long-term service. The buckling due to instability may cause the sudden stiffness reduction of structural members. Therefore, the stability analysis of the reticulated
shell is first carried out in this section to explore the dangerous regions with the possible risk of sudden damage events. After that, the proposed detection approaches are applied to detect the sudden damage events of the reticulated shell.

5.5.1 Stability analysis of reticulated shell

In 1963, a vast reticulated shell in Romania collapsed while it was loaded by locally accumulated snow, at a low exterior temperature and during a snowstorm. The failure examination demonstrated that it was caused by a loss of stability. After that accident, the stability of shell attracts more and more attention across the world (Soare 1963; Beles et al., 1967). The stability analysis of shell is complicated and the nonlinear procedure is commonly adopted to determine the critical loads and buckling properties of the shell. To obtain the load-displacement path, the arch-length method is adopted in this section to analyze the example reticulated shell. The effects of initial geometrical imperfections involving installation errors and connection imperfections are taken into consideration in the analysis. Uniform imperfection mode method proposed by Shen et al (1999) for simulating the random imperfections in structural stability analysis is adopted. Following this method, the eigenvalue buckling analysis is first carried out to obtain the first buckling mode which is then assumed as the imperfection configuration for stability analysis. The maximum value of the first buckling mode is assumed as 1/300 of the shell span. Both eigenvalue buckling analysis and nonlinear instability analysis are carried out by using the commercial software package ANSYS. The eigenvalue buckling problem is formulated as an eigenvalue problem

\[ (\mathbf{K} + \lambda \cdot \mathbf{K}_e)\phi = 0 \]  \hspace{1cm} (5.25)
where $\mathbf{K}$ and $\mathbf{K}_s$ are the stiffness matrix and stress stiffness matrix of shell respectively; $\lambda$ is the eigenvalue; $\mathbf{\phi}$ is the eigenvector. The $\lambda$ in Equation (5.25) is actually the ratio factor to multiply the analytical loads which generates the final theoretical buckling loads. Because eigenvalue buckling analysis does not include the effects of nonlinearity and initial imperfections, it is commonly considered as the upper bound value of actual instability loads. The first eigenvalue buckling mode shape is plotted in Figure 5.22 and the corresponding buckling load is $9.76kN/m^2$.

The instability process of the reticulated shell is computed using nonlinear static analysis incorporating Newton-Raphson iteration. The loads adopted in the stability analysis are the dead loads of the example shell. Six typical nodes 1, 2, 10, 26 50 and 82 attached to radial members in the global $x$ direction are selected to describe their deformation states as shown in Figures 5.23. The load-displacement curve of node 1 (shell vertex) is plotted in Figure 5.24. It can be observed that the relationship between external loads and vertical displacement of node 1 is closely linear during the initial loading stage. The slope of the curve is quite linear which means that the shell is within the elastic stage. Then, the vertical deformation presents obviously nonlinear characteristics when approaching the first peak point. At the first peak point of the curve (Shown in Figure 5.24), the vertical displacement of node 1 is $0.188m$. Further examination reveals that the vertical displacement of node 1 is substantially larger than that of all the other nodes, which actually demonstrates that the first buckling of shell occurs at its vertex. At the first peak of the curve, the slope of the load-displacement curve dramatically reduces to a great extent, which corresponds to the sudden loss of the force bearing capacity of some members within
the first circle. This is actually the sudden damage events due to loss of force bearing capacity caused by the stiffness reduction.

In addition, the variation of displacement of six selected nodes with external loads is listed in Table 5.8 for understanding the collapse process of the reticulated shell. The instability process of the reticulated shell is also plotted in Figure 5.25 to provide the visual view. In Figure 5.25, the Uz denotes the vertical displacement of node 1. It can be observed from the table and figure that if the external loads are relative small (5.895kN/m²), the reticulated shell still works under elastic range and the nodal deformation distribution is basically even. With the increase in external loads (6.741kN/m²), the nodal vertical displacement near the shell vertex firstly presents some nonlinear characteristics while those of other parts of the shell are still within linear range. At the first upper peak point (7.143kN/m²), snap-through instability happens firstly at node 1 and the vertical displacement rapidly increases while simultaneously the vertical displacement of other nodes does not have remarkable variation. It is clear that under the current dead loads, the members connected to node 1 will firstly lose stability. After the snap-through instability of the shell vertex, the load-bearing capacity of the entire shell is dramatically reduced and the nodal vertical displacement substantially increases. If the vertical displacement of node 1 reaches 0.848m, the nodes adjacent to node 1 will have snap-through instability (node 2, 0.187m). With the development of nodal vertical displacement, the number of buckling members rapidly increases and the instability regions rapidly enlarge to cause instability of more nodes. After all the nodes on the circle 3 present snap-through instability, the entire shell loses its load-bearing capacity and eventually
collapses. At this moment, the maximum vertical displacement of shell is about 8.791 m (node 1).

By concluding the collapse process described above, one can understand that on the first upper peak point (7.143 kN/m^2), the first snap-through instability is observed at node 1. After that, the loads present reduction trends until they start second increment steps at local minimum value (4.868 kN/m^2). At this load point, node 1 can be considered reaching opposite stable position. Similar to this process, the nodes on circles 1, 2 and 3 gradually lose stability and the nodal deformation increases until final collapse. It is reported that at the first upper peak point, the maximum vertical deformation of a reticulated shell is commonly no more than 1/300 of its span (Shen et al., 1999). For the reticulated shell investigated in this study, the maximum vertical displacement at the first upper peak point is 0.188 m which is about 1/346 of the shell span. The above discussion is carried out to provide understanding on the whole collapse process of the reticulated shell.

5.5.2 Damage scenarios of reticulated shell

To examine the feasibility of the approaches for identifying sudden damage event of steel space structures, the acceleration responses of the reticulated shell with a sudden change in element stiffness parameter at different members are analyzed. The acceleration responses of the example reticulated shell with a sudden change in element stiffness parameter at different members are analyzed under the El-Centro 1940 earthquake ground acceleration (3 dimensional components) with a peak amplitude 4.0 m/s^2. It should be pointed out that the design peak ground acceleration for extreme earthquake in Shijiazhuang where the reticulated shell is constructed is
0.22g. However, because the example reticulated shell is a kind of Kiewitt shell which has been widely constructed in many places, the design peak ground acceleration of 0.4g is used in the seismic analysis for the region of earthquake intensity 8. The structural parameters of the undamaged shell are the same as those adopted in Chapter 3. The equation of motion of the shell subjected to a sudden reduction of stiffness parameter at the given time instant is then established and it is solved using the Newmark- $\beta$ method with a time step of 0.002s (500Hz sampling frequency). The two factors in the Newmark- $\beta$ method are selected as $\alpha = 1/2$ and $\beta = 1/4$. The Rayleigh damping assumption is adopted to construct the structural damping matrix, and the damping ratios in the first two modes of vibration of the reticulated shell are set as 0.01. As discussed in Chapter 3, the bending responses (bending moments and bending stresses) of shell members under dead loads are quite small compared to axial responses (axial forces and axial stresses). Similar observations can be made from the seismic analysis of the reticulated shell performed in this chapter. To summarise the static and dynamic responses of the shell, one can understand that the major static and dynamic responses of the reticulated shell are observed in member axial direction. The bending responses of members can be neglected in comparison with axial responses. This observation reveals that the axial stiffness of the member is important for the structural force-bearing capacity. Therefore, the stiffness parameter suffered sudden loss during the seismic excitation is selected as the member axial stiffness, namely the product of Young’s modulus and cross section area of member (EA).

The observations made in stability analysis demonstrate that the members within the first circle shall first suffer buckling. Therefore, some members in this region are
selected to simulate the sudden damage events. In the study, only the damage
detection under seismic excitation is conducted. To detect the sudden damage events
of structural members in different positions, three damage scenarios are assumed in
this section as listed in Table 5.9. The location of damaged members is plotted in
Figure 5.26. Listed in Table 5.10 are the first ten natural frequencies of the
reticulated shell without/with sudden damage events. It can be seen that the
frequency reduction due to 40% reduction of EA in a single member is quite small
with a maximum reduction no more than 0.7% in the first 10 natural frequencies.
Following the principle of finite element method, the nodal responses can be
described in both local coordinate system (LCS) and global coordinate system (GCS).
Therefore, the acceleration responses of nodes can be obtained based on both LCS
and GCS. The damage detection based on these two coordinate systems is conducted.

5.5.3 Signal feature due to sudden damage: reticulated shell

The computed acceleration response in local $x$ direction of node 1 of the reticulated
shell for damage scenario 1 (member 1, 40% damage at 4.0 s) is displayed in Figure
5.27 (a) for El Centro earthquake. It is difficult to find the signal feature due to
sudden damage by direct visual inspection of the original acceleration response. The
0.2 second portion of the acceleration response time history, containing the damage
time instant, is expanded in Figure 5.27 (b) to permit a close look at signal feature
due to sudden damage. It can be seen that there exists a sudden jump in the original
signal at the damage time instant. The sudden stiffness reduction of member 1 causes
a clear signal discontinuity in the acceleration response time history in local $x$
direction at the damage time instant. Since the signal discontinuity is of the property
of high frequency, EMD is applied to decompose the original acceleration response
without using intermittency check. As a result, seven IMFs are obtained from the acceleration response time history in local $x$ direction of node 1. Displayed in Figure 5.27 (c) to Figure 5.27 (e) are the first three IMFs of the acceleration response in local $x$ direction of member 1. Similar to the original acceleration response time history, the direct visual inspection of the IMF components cannot find signal features due to sudden damage event. The 0.2 second zooms of the three IMFs around the damage time instant are therefore depicted in Figure 5.27 (f) to Figure 5.27 (h). It can then be seen that the 0.2 second zooms of the first IMF is quite similar to the 0.2 second portion of the original acceleration response time history, and the signal discontinuity is reserved in the first IMF component only. This is because the first IMF contains the highest frequency content of the original acceleration responses. The first three IMFs obtained from the original acceleration response time history in local $y$ and $z$ direction of node 1 and their local zooms around the damage time instant are also plotted in Figure 5.28 and Figure 5.29, respectively. Quite different observations can be made, that is, the local zooms of the first IMF reveal that no obvious signal discontinuity can be inspected in the acceleration responses and corresponding IMFs. The same analysis process is carried out on responses of node 2 which is also attached to the damaged member 1. The analytical results provide the same conclusions from those of node 1. Similarly, the acceleration response and decomposed IMFs of node 1 in global $x$ direction for damage scenario 1 are displayed in Figure 5.30. It can be seen that there exists a sudden jump in the original acceleration response in global $x$ direction at the damage time instant (See Figure 5.30 (b)). This sudden jump can also be observed in the first IMF of the acceleration responses (See Figure 5.30 (f)). The first three IMFs obtained from the original acceleration response time history in global $y$ and $z$
directions of node 1 and their local zooms around the damage time instant are also plotted in Figure 5.31 and Figure 5.32, respectively. No obvious signal discontinuity can be inspected in the acceleration responses and corresponding IMFs. The same conclusions are obtained from the acceleration responses of node 2 in GCS. Therefore, the damage information due to sudden stiffness reduction of shell member has the property of higher frequency and this damage information is reserved in the first IMF of acceleration response time history obtained in both LCS and GCS. Obviously, the signal feature of the reticulated shell due to sudden stiffness reduction is the same as those obtained from the shear building. Therefore, it is reasonable to apply the proposed detection approaches in the damage detection of the reticulated shell with sudden damage event in both LCS and GCS.

5.5.4 Damage time instant and location

Figure 5.33 demonstrates the variation of damage indices with time for different nodes of the reticulated shell for damage scenario 1 using the first approach without EMD. The damage indices are computed using the acceleration responses obtained in the LCS. It is seen that the damage time instant can be easily identified by the occurrence time of the sharp damage indices of node 1 and node 2. The damage indices of nodes 8 and 24 which are not connected by the damaged member are also plotted in Figure 5.33. Obviously, no sharp damage index can be found to explore the sudden damage events. The same observations can also be made from the acceleration responses of other nodes. Therefore, the damage location can also be accurately determined by inspecting the spatial distribution of damage indices in the reticulated shell. Figure 5.34 and Figure 5.35 demonstrate the variation of damage
indices with time for different nodes of the reticulated shell for damage scenario 2 and scenario 3 respectively using the first approach without EMD. Similar to the conclusions obtained in damage scenario 1, the damage time instant can be easily identified by the occurrence time of the sharp damage index. In addition, the damage location can also be accurately determined by inspecting the spatial distribution of damage indices in the reticulated shell. The damage indices for various scenarios are also computed utilizing the second approach with EMD and corresponding results are plotted in Figures 5.36, 5.37 and 5.38 respectively. The damage indices are computed from the first IMF of the original acceleration responses in local $x$ direction. Similar to the observations made without using EMD, the damage time instant can be accurately determined by examining the occurrence time of the sharp damage indices. The examination on spatial distribution of damage indices can accurately explore the location of damage event.

The detection results for damage scenarios 1, 2 and 3 based on acceleration responses obtained in GCS without EMD are plotted in Figures 5.39, 5.40 and 5.41 respectively. Clearly shown in Figure 5.39 (a), damage indices obtained from the acceleration response of node 1 in global $x$ direction form obvious spike at damage instant to reveal the time instant of damage event. The same observations can also be made in damage indices of node 2 in global $x$ direction. While the similar sharp damage indices in global $y$ and $z$ directions cannot be inspected for both node 1 and node 2. For damage scenario 2, the sharp damage indices of node 2 and node 3 in both global $x$ and $y$ directions reveal the sudden damage event at time $t=4.0$ second (See Figure 5.40). In addition, no sharp damage indices in global $z$ direction can be inspected. For damage scenario 3, the sharp damage indices of node 1 and node 3 can
be obtained at time $t=4.0$ second in both global $x$ and $y$ directions to explore the damage instant (See Figure 5.41). Similar to scenario 2, no sharp damage indices can be found in global $z$ direction. The similar analytical procedure is also applied to the acceleration responses of other nodes which are not adjacent to the damaged members for damage scenarios 1, 2 and 3 respectively. The corresponding results demonstrate that no sharp damage indices can be observed in all the three global directions to explore the sudden damage events. Therefore, the damage location can be effectively determined based on the spatial distribution of sharp damage indices. The damage indices for various scenarios are also computed utilizing the second approach with EMD based on acceleration responses in GCS. Similar to the observations made without using EMD, the damage time instant can be accurately determined by examining the occurrence time of the sharp damage indices. The examination on spatial distribution of damage indices can accurately explore the location of damage event. To compare the damage detection on different scenarios based on acceleration responses in GCS, it is seen that the location of damage information in global acceleration responses closely relates to the spatial position of damaged member. The sharp damage indices may possibly exist in acceleration responses in all the three global coordinate directions. Therefore, three accelerometers are required for each member to monitor the possible sudden damage event if adopting the acceleration responses in GCS in detection.

The performance comparison between the different detection processes based on local and global acceleration responses clearly demonstrates the virtues of damage detection process in LCS. This is because the physical parameters assumed to suffer sudden damage is EA which mainly affects the member axial responses (namely the
local $x$ direction). From the viewpoints of practical application, the detection process based on LCS is more convenient and economical because only one accelerometer is required to be placed at the node in member axial direction to monitor the sudden damage event of a certain member. Three accelerometers are required for each node to monitor the sudden damage event of a certain member if the detection process is carried out using acceleration responses in GCS. This conclusion can be adopted to simplify the sensory system for sudden damage detection. Therefore, in the following section, only the detection process based on local acceleration responses is considered to examine the effects of damage severity and noise contamination.

5.5.5 Damage detection on various severities

To further examine the feasibility of the proposed damage index and damage detection approaches, parameter studies are carried out in this section to find the sensitivity of damage index to damage severity. The member 1 attached with nodes 1 and 2 is supposed to suffer four levels of sudden stiffness reduction, 5%, 10%, 20% and 40% but the damage event occurs at the same time (4.0s). The stiffness parameter suffered sudden loss during the seismic excitation is selected as $E_A$. All the parameters for computing acceleration responses of the reticulated shell such as member parameters, external excitation, damping coefficients are the same as those adopted in the former section. The damage indices of the acceleration time history in local $x$ direction of node 1 and node 2 are computed without using EMD and plotted in Figure 5.42 for various damage severities. It is seen that even for small damage event (such as 5% damage), the proposed approach can easily capture the damage time instant without noise contamination. In addition, the examination on spatial distribution of damage indices can accurately explore the location of damage event.
The results in Figure 5.42 demonstrate that the magnitude of sharp damage index also increases with increasing damage severity. The magnitude of sharp damage index corresponding to different damage severities is computed and listed in Table 5.11 for the approaches without/with using EMD respectively. They are also plotted in Figure 5.43 together with a linear fit in which $x$ represents damage severity and $y$ represents damage index. Clearly, the relationship between damage index and damage severity is quite linear for the reticulated shell under seismic excitation. For a given seismic excitation, the larger the damage extent, the larger is the damage index. Also for a given external excitation, the linear relationship obtained by the first approach without using EMD is close to that obtained by the second approach based on the first IMF.

5.5.6 Effects of noise contamination

The effects of measurement noise on detection performance are analyzed in this section. The noise is assumed the same as that utilized for the shear building which includes two noise intensities and three frequency ranges. The detection quality of various damage scenarios by using proposed approaches without/with EMD is investigated. The variation of the damage index for damage scenario 1 is computed by using the first approach without EMD and plotted in Figure 5.44. Clearly, the proposed approach can identify the damage time instant from the contaminated acceleration responses for the two designated noise intensities and three designated noise frequency ranges. Furthermore, by checking the spatial distribution of sharp damage indices within the reticulated shell, the proposed approach can also identify the damage location from the contaminated acceleration responses for the designated noise intensities and frequency ranges. The effects of measurement noise on the
magnitude of damage index are assessed and the results are listed in Table 5.12. It can be seen that as long as the damage event can be identified, the magnitude of damage indices remains almost the same for the designated noise levels and frequency ranges. The similar observations can be made from the results using the second approach with EMD. In addition, the numerical investigation carried out on the damage scenarios 2 and 3 using proposed two approaches demonstrates that the damage time instant, damage location and damage severity can be effectively determined. These observations are not provided here because they are similar to those obtained from damage scenario 1.

The detection observations made in this section indicate that the effects of measurement noise on the magnitude of damage index are small for the reticulated shell. The further numerical simulation indicates that the damage can be effectively identified even with very high noise intensity as long as the noise frequency range is narrow (such as not higher than $50\text{Hz}$). If the noise frequency range is from 0 to $100\text{Hz}$, the reliability of damage detection using the proposed approaches will deteriorate with the increasing noise intensity.

5.6 **COMPARISON WITH WT APPROACH**

To further assess the performance of proposed instantaneous index, the WT approach is applied to the sudden damage detection of the five-storey shear building adopted in this chapter without/with noise contamination. All the damage cases remain the same as those investigated by the proposed instantaneous index. Three noise frequency ranges and two noise intensities, which are the same as those used for assessing the proposed instantaneous index, are repeated. Since the damage detection efficiency of
the WT approach depends on the properties of mother wavelet used, such as, wavelet vanishing moments and support length, three different Daubechies wavelets db1, db2 and db4 are utilized to detect the structural sudden damage of the building subjected to sinusoidal, seismic and impulse excitations, respectively (Hera and Hou 2004). The selected db1, db2 and db4 wavelets have the vanishing moments 1, 2 and 4, respectively, and the corresponding support lengths are 1, 3 and 7, respectively.

The proposed instantaneous index and the WT could deal with the concerned damage detection problem but from different viewpoints. The WT approach decomposes the original signal into several components and detects damage event from the component with the highest frequency. As a result, this approach may encounter some difficulties if the frequency components of an original signal are wide enough to overlap the high frequency damage signal or the energy of damage is too small to be extracted from the original signal. The proposed instantaneous index, on the other hand, detects signal discontinuity at the damage instant directly and it does not need to decompose the signal. Therefore, its detection efficiency is not significantly affected by the high frequency components of the original signal compared with the WT approach.

The numerical results demonstrate that the WT approach using all the three Daubechies wavelets can accurately detect the damage instant of the shear building subjected to sinusoidal excitation. For the seismic excitation case, the WT approach using db1 wavelet fails to detect damage instant while the WT using db2 can detect damage only when the damage extent is large enough. The wavelet coefficient details in level 1 (cD1) obtained by applying the WT approach with db2 and db4 to the
acceleration response time histories at the first floor of the building under seismic excitation are depicted in Figure 5.45 for small damage extent. It can be seen that when the damage extent is small, the db2 wavelet fails to work while the db4 wavelet with longer support length can effectively detect damage. As for the impulse excitation case, the numerical results show that only the WT approach using db4 wavelet can detect the damage instant. This is because the longer the wavelet support length, the finer is the frequency components which is especially useful in detecting high frequency damage signal. Furthermore, the numerical results show that the WT approach using db4 can detect various damage extent of the building under sinusoidal and seismic excitation. However, the satisfactory detection cannot be obtained for impulse excitation case with minor damage extent as shown in Figure 5.46. In reality, for the building subjected to sinusoidal and seismic excitations, the acceleration response spectra have relatively low frequency components but the impulse excitation caused acceleration response spectrum has much wider frequency range which may overlap high frequency damage signal leading to detection fail. Furthermore, the minor damage event may cause small signal energy and conveys minor damage information. This makes the wavelet coefficients from the original signal to the selected mother wavelet be too small to form obvious peaks and reflect damage event. Because of this, the mother wavelets with higher vanishing moments and longer support length (such as db8 or db10) also cannot improve the damage detection efficiency.

The acceleration responses of the shear building with 20% damage at its first floor under the sinusoidal, seismic and impulse excitations are analyzed, respectively, with measurement noise included. The noises are introduced with two noise intensities
and three noise frequency ranges as used before. The results reveal that the WT approach can identify the damage time instant and location from the contaminated acceleration responses for sinusoidal and seismic excitation cases. For impulse excitation case, however, the WT approach fails to identify the damage time instant and damage location when the noise frequency range is from 0 to 250 Hz and the noise intensity is 5%, as shown in Figure 5.47. Obviously, the detection accuracy will decrease with the increase in noise intensity, which is the same as observed when using the proposed instantaneous index.

In summary, the proposed instantaneous index and the WT approach could deal with the sudden damage detection problem but the proposed instantaneous index can provide more accurate results than WT approach in impulse excitation case with measurement noise for small damage events. Furthermore, both the numerical study and the experimental investigation demonstrate that the relationship between damage spike amplitude and damage severity could not be given by WT approach. The proposed instantaneous index, however, could provide a linear relationship between damage index and damage severity. This has been theoretically proved using a SDOF system under impulse excitation. The numerical simulation results from the shear building and the reticulated shell also demonstrate the linear relationship between the damage index and damage severity. The WT approach needs a time history of certain length to carry out the WT and accordingly small time delay is unavoidable in terms of online damage detection. The proposed instantaneous index based on instantaneous index, detects signal discontinuity at the damage instant directly and it does not need to decompose the signal. Therefore, this approach can be implemented in an online health monitoring system without any time delay.
5.7 SUMMARY

The detection of sudden stiffness reduction is carried out in this chapter by proposing an instantaneous damage index to develop effective detection process. The features of signal discontinuity in acceleration response time histories of a structure due to sudden stiffness reduction have been examined and an instantaneous damage index has been proposed. Two damage detection approaches in terms of the proposed damage index have been put forward for the online and offline detection, respectively, of damage time instant, damage location, and damage severity. The first approach can be implemented in an online health monitoring system without any time delay while the second approach needs a time history of certain length to carry out EMD and accordingly small time delay is unavoidable in terms of online damage detection. Nevertheless, the second approach gains an additional insight into the features of sudden damage. Moreover, in conjunction with the Hilbert transform the second approach can provide other information on damage in the time-frequency domain. Extensive numerical simulations have been carried out on the shear building and the reticulated shell to evaluate the performance of proposed damage detection approaches.

The analytical results demonstrate that the proposed two detection approaches can accurately identify the damage time instant and damage location due to a sudden stiffness reduction of the shear building and the reticulated shell in terms of the occurrence time and spatial distribution of sharp damage indices. The proposed damage index is linearly proportional to damage severity but the slope of linear function depends on external excitation and damage time instant. A possible way of determining damage severity has been suggested using the calibrated structural
model and the measured excitation. The proposed two detection approaches both are applicable to the shear building and the reticulated shell. The approach without using EMD is suitable for online health monitoring and damage detection systems while the approach with EMD can provide additional sights on structural damage features in the time-frequency domain in terms of Hilbert transform. The online damage detection defined here means that damage event can be detected without any time delay. The WT and EMD should apply to a time history of certain length, which should then be decomposed into several components. The component with highest frequency can be used for damage detection. Therefore, the WT and EMD detect damage event with certain time delay. Only when this time delay is small, one may also call the WT and EMD online monitoring approaches. The proposed two approaches can effectively identify the damage time instant and damage location from the contaminated acceleration responses. If the noise frequency range is wide enough, the reliability of damage detection using the proposed approaches gradually deteriorates with the increase in noise intensity. As long as the damage event can be identified, the magnitude of damage index remains almost the same for different noise intensities and frequency ranges no matter which approach is used.

The observations made in this chapter demonstrate that the changes in natural frequencies due to sudden stiffness are quite small. Therefore, this damage event cannot be effectively detected by only monitoring the changes in natural frequencies. Alternatively, the accelerometer can be adopted to record the acceleration response with damage information. For the sudden stiffness reduction (EA) of the reticulated shell, only one accelerometer is required for a member to monitor the possible sudden stiffness reduction because the damage information fully exists in the
acceleration responses in member axial direction. The configuration of
collectors will be illustrated in detail in Chapter 9 which discusses the design of
the integrated health monitoring and vibration control system for the reticulated shell.

The sudden stiffness reduction of the structures may be caused by strong external
excitations such as earthquake, blast (impulse excitation) and machine vibration
(sinusoidal excitation). Therefore, only these three kinds of excitations are
considered in this chapter. Normally, the structural vibration due to ambient ground
excitation is small. Thus, the likelihood of sudden stiffness reduction under ambient
ground vibration is very small and this case is not considered. In this chapter, the
shear building and reticulated shell are taken as examples to examine the signal
discontinuity pattern caused by sudden stiffness reduction subjected to three kinds of
external excitations. It should be pointed out that some other structures may
experience a transient high frequency acceleration response due to other reasons than
sudden structural damage, such as a sudden brake of a vehicle in a bridge. In such a
case, a full understanding of the actual reasons and the feature of the associated
signals is necessary before applying the proposed approaches.
### Table 5.1 Natural frequencies without/with sudden damage events

<table>
<thead>
<tr>
<th>Damage extent</th>
<th>$f_1$</th>
<th>$f_2$</th>
<th>$f_3$</th>
<th>$f_4$</th>
<th>$f_5$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1%</td>
<td>2.508 (-0.18%)</td>
<td>7.324 (-0.15%)</td>
<td>11.55 (-0.11%)</td>
<td>14.85 (-0.05%)</td>
<td>16.94 (-0.01%)</td>
</tr>
<tr>
<td>2%</td>
<td>2.504 (-0.36%)</td>
<td>7.313 (-0.30%)</td>
<td>11.54 (-0.21%)</td>
<td>14.84 (-0.11%)</td>
<td>16.94 (-0.03%)</td>
</tr>
<tr>
<td>5%</td>
<td>2.490 (-0.94%)</td>
<td>7.278 (-0.78%)</td>
<td>11.50 (-0.53%)</td>
<td>14.81 (-0.26%)</td>
<td>16.93 (-0.07%)</td>
</tr>
<tr>
<td>10%</td>
<td>2.464 (-1.97%)</td>
<td>7.218 (-1.62%)</td>
<td>11.44 (-1.06%)</td>
<td>14.78 (-0.52%)</td>
<td>16.92 (-0.14%)</td>
</tr>
<tr>
<td>20%</td>
<td>2.407 (-4.41%)</td>
<td>7.088 (-3.48%)</td>
<td>11.32 (-2.18%)</td>
<td>14.71 (-1.01%)</td>
<td>16.90 (-0.26%)</td>
</tr>
<tr>
<td>40%</td>
<td>2.253 (-11.6%)</td>
<td>6.781 (-8.16%)</td>
<td>11.07 (-4.50%)</td>
<td>14.57 (-1.92%)</td>
<td>16.86 (-0.46%)</td>
</tr>
</tbody>
</table>

Note: Values in brackets are the percentage of change in natural frequency.

### Table 5.2 Damage indices around damage time instant

<table>
<thead>
<tr>
<th>Frequency (Hz)</th>
<th>$DI_{i-2}$</th>
<th>$DI_{i-1}$</th>
<th>$DI_{i}$</th>
<th>$DI_{i+1}$</th>
<th>$DI_{i+2}$</th>
<th>$\Delta DI_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Centro earthquake</td>
<td>1.083</td>
<td>160.44</td>
<td>313.97</td>
<td>152.53</td>
<td>0.191</td>
<td>0.32%</td>
</tr>
<tr>
<td>Sinusoidal excitation</td>
<td>0.011</td>
<td>291.58</td>
<td>625.27</td>
<td>329.14</td>
<td>1.223</td>
<td>0.73%</td>
</tr>
<tr>
<td>Impulse excitation</td>
<td>0.690</td>
<td>70.42</td>
<td>152.07</td>
<td>80.78</td>
<td>0.319</td>
<td>0.57%</td>
</tr>
</tbody>
</table>

### Table 5.3 Relationship between damage index and damage severity (without EMD)

<table>
<thead>
<tr>
<th>Damage severity</th>
<th>0%</th>
<th>1%</th>
<th>2%</th>
<th>5%</th>
<th>10%</th>
<th>20%</th>
<th>40%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$DI$ (seismic excitation)</td>
<td>0.001</td>
<td>15.839</td>
<td>31.597</td>
<td>78.868</td>
<td>157.03</td>
<td>315.09</td>
<td>629.72</td>
</tr>
<tr>
<td>$DI$ (sinusoidal excitation)</td>
<td>0.004</td>
<td>31.283</td>
<td>62.560</td>
<td>156.38</td>
<td>312.71</td>
<td>625.27</td>
<td>1249.9</td>
</tr>
<tr>
<td>$DI$ (impulse excitation)</td>
<td>0.002</td>
<td>7.384</td>
<td>15.011</td>
<td>37.890</td>
<td>75.860</td>
<td>152.07</td>
<td>304.57</td>
</tr>
</tbody>
</table>

### Table 5.4 Relationship between damage index and damage severity (with EMD)

<table>
<thead>
<tr>
<th>Damage severity</th>
<th>0%</th>
<th>1%</th>
<th>2%</th>
<th>5%</th>
<th>10%</th>
<th>20%</th>
<th>40%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$DI$ (seismic excitation)</td>
<td>0.002</td>
<td>15.592</td>
<td>31.323</td>
<td>78.270</td>
<td>157.03</td>
<td>313.97</td>
<td>628.23</td>
</tr>
<tr>
<td>$DI$ (sinusoidal excitation)</td>
<td>0.005</td>
<td>31.284</td>
<td>62.559</td>
<td>156.37</td>
<td>312.71</td>
<td>625.27</td>
<td>1249.8</td>
</tr>
<tr>
<td>$DI$ (impulse excitation)</td>
<td>0.001</td>
<td>7.211</td>
<td>14.852</td>
<td>37.714</td>
<td>75.860</td>
<td>152.07</td>
<td>302.42</td>
</tr>
</tbody>
</table>

5-41
Table 5.5 Noise effects on damage index magnitude (seismic excitation)

<table>
<thead>
<tr>
<th>Noise level</th>
<th>Noise frequency range</th>
<th>0~50 Hz</th>
<th>0~100 Hz</th>
<th>0~250 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>without EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>315.09</td>
<td>315.09</td>
<td>315.09</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>315.22 (-0.04%)</td>
<td>317.14 (-0.65%)</td>
<td>322.26 (2.28%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>315.41 (0.10%)</td>
<td>320.22 (1.63%)</td>
<td>333.01 (5.69%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>with EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>313.98</td>
<td>313.98</td>
<td>313.98</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>313.90 (-0.03%)</td>
<td>312.31 (-0.53%)</td>
<td>313.04 (-0.30%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>315.34 (0.43%)</td>
<td>311.21 (-0.88%)</td>
<td>301.55 (-3.95%)</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.6 Noise effects on damage index magnitude (sinusoidal excitation)

<table>
<thead>
<tr>
<th>Noise level</th>
<th>Noise frequency range</th>
<th>0~50 Hz</th>
<th>0~100 Hz</th>
<th>0~250 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>without EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>625.26</td>
<td>625.26</td>
<td>625.26</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>625.00 (-0.04%)</td>
<td>614.57 (-1.71%)</td>
<td>610.84 (-2.31%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>624.02 (-0.20%)</td>
<td>598.55 (-4.27%)</td>
<td>589.21 (-5.76%)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>with EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>625.27</td>
<td>625.27</td>
<td>625.27</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>625.04 (-0.04%)</td>
<td>618.85 (-1.03%)</td>
<td>612.11 (-2.11%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>621.48 (-0.61%)</td>
<td>602.28 (-3.68%)</td>
<td>593.08 (-5.15%)</td>
<td></td>
</tr>
</tbody>
</table>

Table 5.7 Noise effects on damage index magnitude (impulse excitation)

<table>
<thead>
<tr>
<th>Noise level</th>
<th>Noise frequency range</th>
<th>0~50 Hz</th>
<th>0~100 Hz</th>
<th>0~250 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>without EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>152.23</td>
<td>152.23</td>
<td>152.23</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>152.00 (-0.15%)</td>
<td>150.59 (-1.08%)</td>
<td>143.81 (-5.54%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>151.65 (-0.79%)</td>
<td>148.13 (-2.70%)</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>with EMD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No noise</td>
<td>152.073</td>
<td>152.073</td>
<td>152.073</td>
<td></td>
</tr>
<tr>
<td>2% noise</td>
<td>151.83 (-0.15%)</td>
<td>150.37 (-1.10%)</td>
<td>143.70 (-5.51%)</td>
<td></td>
</tr>
<tr>
<td>5% noise</td>
<td>150.87 (-0.79%)</td>
<td>147.94 (-2.70%)</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>
Table 5.8 Vertical displacement of nodes under different load steps (m)

<table>
<thead>
<tr>
<th>Load (kN/m²)</th>
<th>Node 1</th>
<th>Node 2</th>
<th>Node 10</th>
<th>Node 26</th>
<th>Node 50</th>
<th>Node 82</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.859</td>
<td>0.095</td>
<td>0.056</td>
<td>0.083</td>
<td>0.063</td>
<td>-0.076</td>
<td>-0.075</td>
</tr>
<tr>
<td>6.741</td>
<td>0.125</td>
<td>0.062</td>
<td>0.091</td>
<td>0.066</td>
<td>-0.087</td>
<td>-0.093</td>
</tr>
<tr>
<td>7.143</td>
<td><strong>0.188</strong></td>
<td>0.062</td>
<td>0.083</td>
<td>0.064</td>
<td>-0.090</td>
<td>-0.105</td>
</tr>
<tr>
<td>6.977</td>
<td>0.225</td>
<td>0.062</td>
<td>0.077</td>
<td>0.065</td>
<td>-0.088</td>
<td>-0.100</td>
</tr>
<tr>
<td>5.081</td>
<td><strong>0.848</strong>, <strong>0.187</strong></td>
<td>-0.006</td>
<td>0.058</td>
<td>-0.065</td>
<td>-0.061</td>
<td></td>
</tr>
<tr>
<td>0.270</td>
<td>2.225</td>
<td>1.411</td>
<td>-0.020</td>
<td>-0.059</td>
<td>-0.013</td>
<td>0.001</td>
</tr>
<tr>
<td>1.483</td>
<td>3.768</td>
<td>3.047</td>
<td>1.157</td>
<td>-0.234</td>
<td>0.085</td>
<td>-0.011</td>
</tr>
<tr>
<td>3.237</td>
<td>6.446</td>
<td>6.035</td>
<td>4.122</td>
<td>0.862</td>
<td>0.180</td>
<td>-0.007</td>
</tr>
<tr>
<td>1.404</td>
<td>6.922</td>
<td>6.425</td>
<td>4.700</td>
<td>1.600</td>
<td>0.373</td>
<td>0.125</td>
</tr>
<tr>
<td>0.154</td>
<td><strong>8.791</strong></td>
<td>8.159</td>
<td>6.528</td>
<td>3.810</td>
<td>0.037</td>
<td>0.206</td>
</tr>
</tbody>
</table>

Table 5.9 Damage scenarios for the reticulated shell

<table>
<thead>
<tr>
<th>Scenario No.</th>
<th>Damage location</th>
<th>Nodes attached to the damaged member</th>
<th>Damage extent</th>
<th>Damage instant</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1</td>
<td>Member 1</td>
<td>1 and 2</td>
<td>40%</td>
<td>4.0s</td>
</tr>
<tr>
<td>No. 2</td>
<td>Member 49</td>
<td>2 and 3</td>
<td>40%</td>
<td>4.0s</td>
</tr>
<tr>
<td>No. 3</td>
<td>Member 7</td>
<td>1 and 3</td>
<td>40%</td>
<td>4.0s</td>
</tr>
</tbody>
</table>

Table 5.10 Natural frequencies of the reticulated shell without/with sudden damage events

<table>
<thead>
<tr>
<th>Freq. No.</th>
<th>Original</th>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>f₁</td>
<td>4.0617</td>
<td>4.0617 (0.000%)</td>
<td>4.0483(-0.3299%)</td>
<td>4.0617 (0.0000%)</td>
</tr>
<tr>
<td>f₂</td>
<td>4.0629</td>
<td>4.0629 (0.0000%)</td>
<td>4.0626(-0.0073%)</td>
<td>4.0629 (0.0000%)</td>
</tr>
<tr>
<td>f₃</td>
<td>4.3897</td>
<td>4.3888 (-0.0194%)</td>
<td>4.3629(-0.6105%)</td>
<td>4.3828 (-0.1553%)</td>
</tr>
<tr>
<td>f₄</td>
<td>4.4005</td>
<td>4.3904 (-0.2294%)</td>
<td>4.3932(-0.1658%)</td>
<td>4.4001 (-0.0078%)</td>
</tr>
<tr>
<td>f₅</td>
<td>4.4104</td>
<td>4.4099 (-0.0129%)</td>
<td>4.4012(-0.2086%)</td>
<td>4.4068 (-0.0832%)</td>
</tr>
<tr>
<td>f₆</td>
<td>4.4171</td>
<td>4.4164 (-0.0171%)</td>
<td>4.4112(-0.1335%)</td>
<td>4.4162 (-0.0222%)</td>
</tr>
<tr>
<td>f₇</td>
<td>4.4256</td>
<td>4.4220 (-0.0801%)</td>
<td>4.4236(-0.0451%)</td>
<td>4.4236 (-0.0450%)</td>
</tr>
<tr>
<td>f₈</td>
<td>4.4269</td>
<td>4.4269 (-0.0000%)</td>
<td>4.4253(-0.0361%)</td>
<td>4.4256 (-0.0299%)</td>
</tr>
<tr>
<td>f₉</td>
<td>4.4758</td>
<td>4.4716 (-0.0959%)</td>
<td>4.4747(-0.0245%)</td>
<td>4.4720 (-0.0849%)</td>
</tr>
<tr>
<td>f₁₀</td>
<td>4.4773</td>
<td>4.4769 (-0.0096%)</td>
<td>4.4769(-0.0089%)</td>
<td>4.4759 (-0.0316%)</td>
</tr>
</tbody>
</table>

Note: Values in brackets are the percentage of change in natural frequency
Table 5.11 Relationship between damage index and damage severity of the reticulated shell

<table>
<thead>
<tr>
<th>Damage severity</th>
<th>0%</th>
<th>5%</th>
<th>10%</th>
<th>20%</th>
<th>40%</th>
</tr>
</thead>
<tbody>
<tr>
<td>DI of node 1 (without EMD)</td>
<td>0.002</td>
<td>761.86</td>
<td>1522.0</td>
<td>3042.8</td>
<td>6082.3</td>
</tr>
<tr>
<td>DI of node 1 (with EMD)</td>
<td>0.002</td>
<td>763.73</td>
<td>1523.4</td>
<td>3046.1</td>
<td>6089.3</td>
</tr>
<tr>
<td>DI of node 2 (without EMD)</td>
<td>0.005</td>
<td>1079.9</td>
<td>2163.6</td>
<td>4332.3</td>
<td>8669.1</td>
</tr>
<tr>
<td>DI of node 2 (with EMD)</td>
<td>0.003</td>
<td>1091.1</td>
<td>2184.8</td>
<td>4372.2</td>
<td>8742.9</td>
</tr>
</tbody>
</table>

Table 5.12 Noise effects on damage index magnitude of the reticulated shell
(Damage scenario 1, without EMD)

<table>
<thead>
<tr>
<th>Noise level</th>
<th>Noise frequency range</th>
<th>0~50 Hz</th>
<th>0~100 Hz</th>
<th>0~250 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>No noise</td>
<td></td>
<td>6082.3</td>
<td>6082.3</td>
<td>6082.3</td>
</tr>
<tr>
<td>2% noise</td>
<td></td>
<td>6081.1</td>
<td>6090.1</td>
<td>6098.3</td>
</tr>
<tr>
<td>5% noise</td>
<td></td>
<td>6079.2</td>
<td>6101.7</td>
<td>6122.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Noise level</th>
<th>Noise frequency range</th>
<th>0~50 Hz</th>
<th>0~100 Hz</th>
<th>0~250 Hz</th>
</tr>
</thead>
<tbody>
<tr>
<td>No noise</td>
<td></td>
<td>8669.1</td>
<td>8669.1</td>
<td>8669.1</td>
</tr>
<tr>
<td>2% noise</td>
<td></td>
<td>8670.2</td>
<td>8661.6</td>
<td>8653.8</td>
</tr>
<tr>
<td>5% noise</td>
<td></td>
<td>8672.0</td>
<td>8650.6</td>
<td>8630.9</td>
</tr>
</tbody>
</table>
Figure 5.1 Acceleration response and its IMFs of the SDOF system.

Figure 5.2 Acceleration response and its slopes of a SDOF system (impulse excitation).
Figure 5.3 Elevation of a five-storey building model.

Figure 5.4 Signal discontinuity due to sudden damage (seismic excitation).
Figure 5.5 Signal discontinuity due to sudden damage (sinusoidal excitation).

Figure 5.6 Signal discontinuity due to sudden damage (impulse excitation).
Figure 5.7 Signal discontinuity patterns around damage time instant.

(a) Slope +++ ++ (b) Slope +++ -- (c) Slope ---- (d) Slope -- ++

(e) Slope -- ++ (f) Slope -- + + (g) Slope + + + + (h) Slope + + + --

Figure 5.8 Damage index patterns around damage time instant:
(a) $DI_{t+1} < DI_{t+1}$; (b) $DI_{t+1} > DI_{t+1}$.
Figure 5.9 Two detection approaches for sudden damage.

Figure 5.10 Variation of damage index with time (seismic excitation).
Figure 5.11 Variation of damage index with time (sinusoidal excitation).
Figure 5.12 Variation of damage index with time (impulse excitation).
Figure 5.13 Sensitivity of damage index to damage severity (seismic excitation):
(a) Signal discontinuity of the first IMF; (b) Damage index of the first floor.
Figure 5.14 Sensitivity of damage index to damage severity (sinusoidal excitation):
(a) Signal discontinuity of the first IMF; (b) Damage index of the first floor.
Figure 5.15 Sensitivity of damage index to damage severity (impulse excitation): (a) Signal discontinuity of the first IMF; (b) Damage index of the first floor.

Figure 5.16 Improved minor damage detection (impulse excitation).
Figure 5.17 Relationship between damage index and damage severity:
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Figure 5.18 First IMF components of contaminated acceleration responses with sudden damage (seismic excitation): (a) Noise frequency range 0~50 Hz; (b) Noise frequency range 0~100 Hz; (c) Noise frequency range 0~250 Hz.

Figure 5.19 Damage index from contaminated acceleration responses with sudden damage (seismic excitation): (a) Noise frequency range 0~50 Hz; (b) Noise frequency range 0~100 Hz; (c) Noise frequency range 0~250 Hz.
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damage (sinusoidal excitation): (a) Noise frequency range 0–50 Hz; (b) Noise
frequency range 0–100 Hz; (c) Noise frequency range 0–250 Hz.

Figure 5.21 Damage index from contaminated acceleration responses with sudden
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Figure 5.23 Number of nodes of the reticulated shell.
Figure 5.24 The load-displacement curve for node 1.
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Figure 5.27 Signal discontinuity due to sudden damage (node 1, local x direction).

Figure 5.28 Signal discontinuity due to sudden damage (node 1, local y direction).
Figure 5.29 Signal discontinuity due to sudden damage (node 1, local \( z \) direction).

Figure 5.30 Signal discontinuity due to sudden damage (node 1, global \( x \) direction).
Figure 5.31 Signal discontinuity due to sudden damage (node 1, global y direction).

Figure 5.32 Signal discontinuity due to sudden damage (node 1, global z direction).
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Figure 5.34 Variation of damage index with time (scenario 2, without EMD, LCS):
(a) Node 2; (b) Node 3.

Figure 5.35 Variation of damage index with time (scenario 3, without EMD, LCS):
(a) Node 1; (b) Node 3.
Figure 5.36 Variation of damage index with time (scenario 1, with EMD, LCS):
(a) Node 1; (b) Node 2.

Figure 5.37 Variation of damage index with time (scenario 2, with EMD, LCS):
(a) Node 2; (b) Node 3.
Figure 5.38 Variation of damage index with time (scenario 3, with EMD, LCS):
(a) Node 1; (b) Node 3.

Figure 5.39 Variation of damage index with time (scenario 1, without EMD, GCS):
(a) Node 1; (b) Node 2.
Figure 5.40 Variation of damage index with time (scenario 2, without EMD, GCS):
(a) Node 2; (b) Node 3.

Figure 5.41 Variation of damage index with time (scenario 3, without EMD, GCS):
(a) Node 1; (b) Node 3.
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Figure 5.46 Damage detection using db4 wavelet under impulse excitation.
Figure 5.47 Detection results from contaminated acceleration responses using db4 wavelet (impulse excitation): (a) Noise frequency range 0–50 Hz; (b) Noise frequency range 0–100 Hz; (c) Noise frequency range 0–250 Hz.
CHAPTER 6

INTEGRATED HEALTH MONITORING AND VIBRATION CONTROL

6.1 INTRODUCTION

Vibration control and health monitoring of civil engineering structures under harsh environment have been actively investigated in recent years. In the aspect of vibration control, several technologies have been developed in designing civil engineering structures to reduce excessive vibration caused by strong winds, severe earthquakes or other disturbances (Housner et al., 1997; Spencer et al., 2003). Passive control technology has been evolved into a workable technology and implemented in real structures but its control performance is always limited because of its passive nature. Active/hybrid control technology has also been adopted in several structures but cost effectiveness and reliability consideration have limited their wide acceptance in civil engineering. Semi-active control technology is now receiving considerable attention from engineering professionals because it offers the reliability of passive control systems and at the same time maintains the versatility and adaptability of active/hybrid control systems with much lower power requirement which is a critical issue during earthquakes for operating active/hybrid control systems. To put semi-active control technology into effect, sensory system and data acquisition and transmission system are required in addition to semi-active control devices and control algorithm.
In the aspect of health monitoring, monitoring systems consisting of sensors, data acquisition and data analysis have been developed and implemented in designing civil engineering structures for the identification of their dynamic characteristics and parameters and for the detection of possible damage after extreme event or long-term service (Aktan et al., 2000; Wong et al., 2000). System identification is defined as the process of developing or improving a mathematical model of a physical system using measurement data to describe the input, output and noise relationship. The first step towards this goal is the development of a mathematical model to represent the physical system. The second step consists in the estimation of the parameters involved in the mathematical model so that the mathematical model can best reproduce the physical system. In civil engineering, various methods have been developed to improve the quality of the finite element model of a structure using measurement data. A detailed discussion of system identification methods can be found in a review paper by Mottershead and Firswell (1993). Structural damage can be defined as changes in structural parameters which adversely affect the current or future performance of the structure whereas structural damage detection aims to find such changes in the structure using measurement data. Vibration-based structural damage detection methods have attracted considerable attention in recent years for the assessment of health and safety of civil engineering structures. Most of currently-used vibration-based structural damage detection methods are built on the idea that the measured modal parameters or the properties derived from these modal parameters are functions of the physical properties of the structure and, therefore, changes in the physical properties will cause detectable changes in the model parameters. A detailed discussion of vibration-based damage detection methods can be found in a review paper by Dowbling et al (1998).
Although vibration control system and health monitoring system both need sensors, data acquisition and transmission for real implementation, in most of the investigations vibration control and health monitoring have been treated separately according to the primary objective pursued. This separate approach is not practical and cost-effective if structures do require both vibration control system and health monitoring system. This separate approach is also not beneficial for the utmost goal of creating smart structures with their own sensors (nervous system), processor (brain system), and actuators (muscular system)-thus mimicking biological systems. In this regard, Ray and Tian (1999) proposed a method of enhancing modal sensitivity to local damage using feedback control to aid in damage detection. Gattulli and Romeo (2000) proposed the use of an integrated procedure for robust control of oscillations and damage detection of linear structural systems. Sun and Tong (2003) presented a closed-loop control based damage detection scheme aiming at detecting small damage in controlled structures. These studies, however, focus on actively controlled mechanical systems or small structures, which may be difficult to be applied to civil engineering structures.

The detection of sudden damage events is conducted by using two approaches based on the proposed damage index in Chapter 5 respectively. These detection approaches which are conducted based on time histories of structural responses can capture the damage information at damage instant to explore the structural behavior during the occurrence of the damage event. Besides the structural performance at damage instant, the changes of structural properties (such as natural frequencies and mode shapes) before and after the damage events are also useful for the safety assessment of the structure without considering details at damage instant. This has attracted
many attentions in recent years. Many detection approaches such as vibration-based damage detection has been developed in order to identify damage through the understanding of the structural properties before and after damage events (Sohn et al., 2003), which is also one of the objectives in this study.

This chapter presents an integrated procedure for vibration control and health monitoring of building structures using semi-active friction dampers. The concept of the integrated system using semi-active friction dampers is firstly introduced. A model updating scheme based on adding known stiffness by using semi-active friction dampers is then presented to update the structural stiffness and mass matrices and to identify its structural parameters. Based on the updated system matrices, the vibration control performance of semi-active friction dampers is investigated using a shear building subject to earthquake excitation. By assuming that the structures suffer certain damage after extreme event or long-term service and by using the previously identified original structural parameters, a damage detection scheme based on adding known stiffness using semi-active friction dampers is finally proposed and used for damage detection. The feasibility and accuracy of the proposed integrated procedure is demonstrated through detailed numerical example and parameter studies. The observations made in this chapter will provide many constructive guidelines for the application of integrated monitoring and control system for other complicated structures. As a result, the integrated system using semi-active friction dampers will be applied to the reticulated shell in the next chapter for vibration control, model updating and damage detection.
6.2 INTEGRATED VIBRATION CONTROL AND HEALTH MONITORING SYSTEM

6.2.1 Vibration control system using semi-active friction dampers

Let us consider a multi-storey shear building subjected to earthquake excitation, as shown in Figure 6.1 (a). To reduce possible excessive vibration caused by earthquake, either passive or semi-active dampers can be positioned with diagonal braces into the building storeys to enhance its vibration energy dissipation capacity. If semi-active control systems are used, the type, location and number of semi-active dampers should be firstly determined for the concerned building. The sensory system and the data acquisition and transmission system should then be designed properly to provide correct and complete feedback information to the control system in which the feedback information is processed according to the control algorithm to determine optimal control signals. The optimal control signals are finally sent to the semi-active dampers to change their parameters to achieve optimal control forces by which the maximum building response reduction can be achieved.

Semi-active friction dampers are considered in this study for integrated vibration control and health monitoring of a building. A semi-active friction damper together with a diagonal brace can be basically modeled with the components of a linear spring and a variable friction slider connected in series, as shown in Figure 6.1 (b). The mechanical behavior of friction slider is assumed to effectively follow the typical Coulomb friction model of which the constant dynamic friction coefficient is considered independent of sliding velocity and displacement. The friction force of the damper is therefore linearly proportional to the clamping force. By incorporating a piezoelectronic actuator, for instance, into the damper to manipulate clamping force
according to a feedback controller, the friction force of the damper can be actively adjusted and the seismic response of the building can be reduced accordingly (Chen and Chen 2002; Ng and Xu 2004). In particular, when a clamping force is set at zero the stiffness of diagonal brace with the friction damper is also zero, and thus no additional stiffness is provided for the building. If clamping force is set at such a value that the friction damper is always in sticking state, the stiffness of diagonal brace will then provide an additional known stiffness for the building. The modal updating scheme and the damage detection scheme which will be proposed in this study are actually based on the preceding two states.

6.2.2 Health monitoring system

To identify dynamic characteristics and parameters and to detect possible damage of a building after extreme events or long-term service, the sensory system and the data acquisition and transmission system should be designed properly to provide correct and complete information for system identification and damage detection, as shown in Figure 6.2. Then, the selection of proper system identification algorithm and damage detection algorithm become a challenging task. This is because in most of the currently-used modal updating methods there exists one or more of the following shortcomings (Cha and de Pillis 2001): (1) give fully populated mass and stiffness matrices and thus fail to preserve the physical connectivity of the system; (2) accommodate only when the deviations of the actual parameters from the analytical values are small; and (3) require that the measured modal matrix be properly normalized with respect to the actual mass and stiffness matrices which are not known a priori. This is also because most of the currently-used vibration-based damage detection methods presume access to the system matrices and/or the
structural parameters of the undamaged structure, which are often not available. Furthermore, by comparing the vibration control system shown in Figure 6.1 (a) with the health monitoring system shown in Figure 6.2 one can see that both vibration control system and health monitoring system need the sensory system and the data acquisition and transmission system. From a practical point of view, it is desirable to have the common sensory system and the common data acquisition and transmission system for both vibration control and health monitoring of the building.

6.2.3 Integrated vibration control and health monitoring system

This study thus aims to present an integrated vibration control and health monitoring system for building structures to fulfill modal updating, seismic response mitigation, and damage detection in a systematic and interactive way. The first task is to present a model updating scheme to update the stiffness and mass matrices of a building and to identify its structural parameters based on adding known stiffness using semi-active friction dampers to create the two states for the building: (1) the original building without any additional stiffness (clamping force is set at zero); and (2) the original building with additional stiffness (damper is in sticking state). This model updating scheme can avoid some shortcomings existing in the currently-used methods. The updated system matrices and structural parameters will facilitate the implementation of structural vibration control and provide the reference for the subsequent damage detection.

The second task is to present a local feedback control algorithm with a Kalman filter for the building with semi-active friction dampers subjected to earthquake excitation. The primary purpose of implementing local feedback control is to involve minimal
feedback signals to achieve reliable and economical control design while the use of
the Kalman filter makes it possible to use the accelerometers as sensors for both
health monitoring and vibration control, leading to the common sensory system and
the common data acquisition and transmission system. Nevertheless, the feedback
signal is localized and the capacity of providing a full picture of vibration level of the
entire building is limited. Thus, there is a need to extensively investigate the control
effectiveness of local-feedback control in comparison with the global feedback
control counterpart.

The last task is to apply the proposed modal updating scheme to the building with
damage to identify the structural parameters of the damaged building based on
adding known stiffness using semi-active friction dampers to create the two states: (1)
the damaged building without any additional stiffness (clamping force is set at zero);
and (2) the damaged building with additional stiffness (damper is in sticking state).

By comparing with the structural parameters of the undamaged structure, the location
and severity of the structural damage can be determined. Figure 6.3 shows a
schematic diagram for the proposed integrated vibration control and health
monitoring system for a shear building. The details of each task will be introduced in
the subsequent sections.

6.3 PARAMETER IDENTIFICATION OF ORIGINAL BUILDING

The modal updating by adding known masses, proposed by Cha and de Pillis (2001),
has been extended in this study to update not only the stiffness and mass matrices of
a building structure but also to identify its structural parameters by adding known
stiffness using semi-active friction dampers based on the two states of the building:
(1) the original building without any additional stiffness (clamping force is set at zero); and (2) the original building with additional stiffness (damper is in sticking state). This is because adding known masses for modal updating is not a favorable scheme for most of civil engineering structures, and the identification of structural parameters first and then the construction of stiffness and mass matrices using such a scheme are more feasible and the results are more accurate. The eigenvalue problem of the original building structure with \( N \) degrees of freedom without any additional stiffness can be given by

\[
K_0X_0 = M_0X_0\Lambda_0
\]

(6.1)

where \( K_0 \) and \( M_0 \) are, respectively, the stiffness and mass matrices of the original building structure; \( X_0 \) is the modal matrix of the original building structure; and \( \Lambda_0 \) is the diagonal matrix whose elements are related to the natural frequencies of the original building structure. The objective of system identification is to identify the mass and stiffness matrices of the building structure using the measured modal shapes and natural frequencies.

Assume that the semi-active friction dampers installed in the building are so designed that all semi-active friction dampers are in sticking state when taking measurement of the building under ambient excitation to find its modal shapes and natural frequencies and that the additional stiffness matrix attributed by the braces is known and denoted as \( K_a \). Then, the eigenvalue problem of the building structure with the additional stiffness can be given by

\[
(K_0 + K_a)X = M_0X\Lambda
\]

(6.2)

where \( X \) is the modal matrix of the building structure with additional stiffness; and \( \Lambda_0 \) is the diagonal matrix whose elements are related to the natural frequencies of
the building structure with additional stiffness. In reality, it is often difficult to measure all natural frequencies and modal shapes of a building structure. By assuming that the available number of measured natural frequencies and model shapes is $N_e$ at all degrees of freedom, Equations (6.1) and (6.2) can be modified as

$$\mathbf{K}_0 \mathbf{X}_{0k} = \mathbf{M}_0 \mathbf{X}_{0k} \Lambda_{0k}$$  \hspace{1cm} (6.3)

$$\left(\mathbf{K}_0 + \mathbf{K}_a\right) \mathbf{X}_k = \mathbf{M}_0 \mathbf{X}_k \Lambda_k$$  \hspace{1cm} (6.4)

where $\mathbf{X}_{0k}$, $\mathbf{X}_k$ are the measured $N \times N_e$ modal matrix of the building structure without and with additional stiffness respectively; and $\Lambda_{0k}$, $\Lambda_k$ are the $N_e \times N_e$ diagonal matrix whose elements are related to the measured $N_e$ natural frequencies of the building structure without and with additional stiffness respectively. Taking the transpose of Equation (6.3), postmultiplying the resulting matrix equation by $\mathbf{X}_k$ and then premultiplying the resulting matrix equation by $\Lambda^{-1}_0$ yield

$$\Lambda^{-1}_0 \mathbf{X}^T_{0k} \mathbf{K}_0 \mathbf{X}_k = \mathbf{X}^T_{0k} \mathbf{M}_0 \mathbf{X}_k$$ \hspace{1cm} (6.5)

Premultiplying Equation (6.4) by $\mathbf{X}^T_{0k}$ and then postmultiplying the resulting matrix equation by $\Lambda^{-1}_k$ yield

$$\mathbf{X}^T_{0k} \left(\mathbf{K}_0 + \mathbf{K}_a\right) \mathbf{X}_k \Lambda^{-1}_k = \mathbf{X}^T_{0k} \mathbf{M}_0 \mathbf{X}_k$$ \hspace{1cm} (6.6)

Subtracting Equation (6.6) from (6.5) yields

$$\Lambda^{-1}_0 \mathbf{X}^T_{0k} \mathbf{K}_0 \mathbf{X}_k - \mathbf{X}^T_{0k} \mathbf{K}_0 \mathbf{X}_k \Lambda^{-1}_k = \mathbf{S}_k$$ \hspace{1cm} (6.7)

in which

$$\mathbf{S}_k = \mathbf{X}^T_{0k} \mathbf{K}_a \mathbf{X}_k \Lambda^{-1}_k$$ \hspace{1cm} (6.8)

Let

$$\mathbf{U}_k = \mathbf{X}^T_{0k} \mathbf{K}_a \mathbf{X}_k$$ \hspace{1cm} (6.9)

Equation (6.7) can be simplified as
\[ \mathbf{A}_{0k}^{-1} \mathbf{U}_k - \mathbf{U}_k \mathbf{A}_k^{-1} = \mathbf{S}_k \]  

(6.10)

Considering that both \( \mathbf{A}_k \) and \( \mathbf{A}_{0k} \) are diagonal matrices, Equation (6.10) can be expanded so that its \((i,j)\)th element is given by

\[
\left( \frac{1}{\lambda_{0kj}} - \frac{1}{\lambda_{kj}} \right) U_{kij} = S_{kij} \quad (i, j = 1, 2, ... N_e) 
\]

(6.11)

According to Equation (6.8), the element \( S_{kij} \) can be determined by the known additional stiffness matrix, the measured modal shapes of the building structure without additional stiffness, and the measured modal shapes and natural frequencies of the building structure with additional stiffness. Therefore, if the measured natural frequencies of the building structure without and with additional stiffness do not coincide, all unknowns \( U_{kij} \) can be found using Equation (6.11) and then the matrix \( \mathbf{U}_k \) can be determined. Because both \( \mathbf{X}_k \) and \( \mathbf{X}_{0k} \) in Equation (6.9) are rectangular matrices, they have no inverses. Therefore, to find \( \mathbf{K}_0 \) Equation (6.9) is rewritten so that \( \mathbf{K}_0 \) appears as an unknown column vector \( \mathbf{k}_0 \) as follows:

\[
\mathbf{A}_k \cdot \mathbf{k}_0 = \mathbf{h}_k 
\]

(6.12)

where

\[
\mathbf{A}_k = \mathbf{X}_{0k}^T \otimes \mathbf{X}_k^T 
\]

(6.13)

\[
\mathbf{k}_0 = [k_{i1} \cdots k_{iN} \mid k_{21} \cdots k_{2N} \mid \cdots \mid k_{N1} \cdots k_{NN}]^T 
\]

(6.14)

\[
\mathbf{h}_k = [h_{i1}^k \cdots h_{iN}^k \mid h_{21}^k \cdots h_{2N}^k \mid \cdots \mid h_{N1}^k \cdots h_{NN}^k]^T 
\]

(6.15)

in which \( \mathbf{A}_k \) is the coefficient matrix of size \( N_c^2 \times N_c^2 \) defined in Equation (6.13) in which \( \otimes \) is the Kronecker product; the vector \( \mathbf{k}_0 \) is of length \( N_c^2 \) and its element \( k_{ij} \) corresponds to the \((i,j)\)th element of \( \mathbf{K}_0 \); and the vector \( \mathbf{h}_k \) is of length \( N_c^2 \) and its
element $h^k_{ij}$ is equal to $U_{kij}$. Equation (6.12) is named as the identification equation for stiffness matrix elements.

If all the natural frequencies and modal shapes can be measured ($N_e = N$), Equation (6.12) can be solved exactly by using simple Gaussian elimination. If $N_e < N$, Equation (6.12) yields an underdetermined problem and will have an infinite number of solutions. The numerical investigation in this study reveals that the minimum norm least-squares solution to the underdetermined problem expressed by Equation (6.12) is often not accurate. Furthermore, the minimum norm least-squares solution to Equation (6.12) will fail to preserve the sparsity and symmetric properties of the analytical stiffness matrix because the stiffness matrix of most engineering structures has many zero elements in consideration of the connectivity information of structural members. The sparsity information should thus be taken as constraint conditions to the updated system matrix. Mathematically, this can be achieved by eliminating all the known zero elements from $k_o$ and by deleting all the corresponding columns in matrix $A_k$. For the shear building considered in this study, the stiffness matrix is tridiagonal and $k_{ij} = 0$ ($\text{abs}(i - j) > 1$). Equation (6.12) can be simplified as

$$A_k^m \cdot k^m_0 = h_k$$

in which

$$k^m_0 = [k_{i1}, k_{i2}, ..., k_{ij}, ..., k_{NN}]^T \quad (\text{abs}(i - j) \leq 1)$$

$A_k^m$ is obtained from $A_k$ by deleting all the columns that multiply by $k_{ij} = 0$ ($\text{abs}(i - j) > 1$). In such a way, the vector $k_o$ of size $N^2 \times 1$ is reduced to $k^m_0$ of size $(3N - 2) \times 1$. 

6-12
In most cases, the identification equation, Equation (6.16), is still underdetermined due to the limited number of measured modal shapes, and the identified results may be still questionable. Moreover, the identified results from Equation (6.16) are the elements of stiffness matrix other than structural parameters. Thus, the identified results from Equation (6.16) cannot be directly used for damage detection. In this regard, a transformation matrix is introduced in this study to overcome the aforementioned shortcomings. For an \( N \)-storey shear building, the number of unknown horizontal storey stiffness coefficients is \( N \) whereas the number of nonzero elements in the stiffness matrix is \((3N-2)\). The relationship between the horizontal storey stiffness coefficients \( k_i(i=1,2,\ldots,N) \) and the elements of stiffness matrix can be established as

\[
\mathbf{k}_0^m = \mathbf{T}\mathbf{k}_0^p
\]  

(6.18)

\[
\mathbf{k}_0^p = [k_1, k_2, \ldots, k_N]^T
\]  

(6.19)

\[
\mathbf{T} = \begin{bmatrix}
    a_{11} & a_{12} & \cdots & a_{1N} \\
    a_{21} & a_{22} & \cdots & a_{2N} \\
    \vdots & \vdots & \ddots & \vdots \\
    a_{(3N-2)1} & a_{(3N-2)2} & \cdots & a_{(3N-2)N}
\end{bmatrix}
\]  

(6.20)

where the vector \( \mathbf{k}_0^p \) is the horizontal storey stiffness coefficient vector of the original building structure; and the matrix \( \mathbf{T} \) is the transformation matrix of size \((3N-2)\times N\).

The substitution of Equation (6.18) into Equation (6.16) yields the identification equation as

\[
(\mathbf{A}_x^m\mathbf{T})\mathbf{k}_0^p = \mathbf{h}_x
\]

(6.21)

If \( N_e^2 = N \), Equation (6.21) can be solved exactly to give the horizontal storey stiffness coefficient vector. If \( N_e^2 > N \) or \( N_e^2 < N \), Equation (6.21) yields an
overdetermined or underdetermined problem. The minimum norm least-squares solution is given by

$$ k_0^p = (A_0^m T)^+ h_k $$  \hspace{1cm} (6.22)

in which $ (\quad)^+$ is the matrix pseudoinverse. Once $ k_0^p $ is identified from Equation (6.22), the vector $ k_0^m $ can be determined using Equation (6.18) and the matrix $ K_0 $ can then be constituted.

The mass matrix of the original building structure can also be identified using a similar procedure. From Equations (6.3) and (6.4), the following equation can be derived.

$$ X_{0k}^T M_{0k} X_k \Lambda_k - \Lambda_{0k} X_{0k}^T M_{0k} X_k = Q_k $$ \hspace{1cm} (6.23)

where

$$ Q_k = X_{0k}^T K_a X_k $$ \hspace{1cm} (6.24)

Let

$$ P_k = X_{0k}^T M_{0k} X_k $$ \hspace{1cm} (6.25)

Equation (6.23) can be simplified as

$$ P_k \Lambda_k - \Lambda_{0k} P_k = Q_k $$ \hspace{1cm} (6.26)

Considering both $ \Lambda_k $ and $ \Lambda_{0k} $ are diagonal matrices, Equation (6.26) can be easily expanded so that its $ (i,j) $th element yields

$$ (\lambda_{ik} - \lambda_{0ik}) P_{kij} = Q_{kij} \quad (i,j = 1,2,\ldots N_e) $$ \hspace{1cm} (6.27)

According to Equation (6.24), the element $ Q_{kij} $ can be determined by the known additional stiffness matrix, the measured modal shapes of the building structure with and without additional stiffness. Therefore, if the measured natural frequencies of the
building structure without and with additional stiffness do not coincide, all unknowns $P_{ij}$ can be found using Equation (6.27) and then the matrix $P_k$ can be determined.

Similar to Equation (6.9), the identification equation for mass matrix, Equation (6.25) can be expressed as

$$A_k \cdot m_0 = h_m$$  \hspace{1cm} (6.28)

where

$$m_0 = [m_1 \cdots m_{1N} | m_{21} \cdots m_{2N} | \cdots | m_{N1} \cdots m_{NN}]^T$$  \hspace{1cm} (6.29)

$$h_m = [h_{1}^{m} \cdots h_{1N}^{m} | h_{21}^{m} \cdots h_{12N}^{m} | \cdots | h_{NN}^{m} \cdots h_{NN}^{m}]^T$$  \hspace{1cm} (6.30)

Furthermore, the sparsity information of the mass matrix can be applied to reduce the size of the identification equation for mass matrix. For the shear building considered in this study, the analytical mass matrix is a diagonal matrix and $m_{ij} \neq 0$.

Equation (6.28) can be simplified as

$$A_m^m \cdot m_0^m = h_m$$  \hspace{1cm} (6.31)

in which

$$m_0^m = [m_{1i} \cdots m_{N_i}]^T \hspace{1cm} (i = 1, 2, \ldots, N)$$  \hspace{1cm} (6.32)

$A_m^m$ is obtained from $A_k$ by deleting all the columns which multiply by $m_{ij} = 0$.

As a result, the vector $m_0$ of size $N^2 \times 1$ is reduced to the vector $m_0^m$ of size $N \times 1$. The minimum norm least-squares solution of Equation (6.31) then gives the mass matrix $M_0$ of the original building structure.

As for the damping matrix of the original building structure, the practical approach is to identify the modal damping ratios from the measured structural responses and then
construct the damping matrix $C_0$ based on the Rayleigh damping assumption. The flow chart of the proposed system identification process is presented in Figure 6.4.

6.4 VIBRATION CONTROL OF ORIGINAL BUILDING

6.4.1 Modeling of building with semi-active friction dampers

A number of control strategies for semi-active friction dampers have been proposed, which can be classified into two types: (i) the global feedback control strategies which use clipped strategies to let semi-active friction dampers work effectively with the linear quadratic Gaussian (LQG) controller (Ng and Xu 2004); (ii) the local feedback control strategies which use local motions as feedback signals to change damper clamping forces in such a way to let the friction dampers slip as much as possible in order to achieve maximum energy dissipation (Chen and Chen 2002; Ng and Xu 2004). In most of existing local feedback control strategies, velocity and displacement feedback signals other than acceleration feedback signals are used. However, accelerometer is the mostly-used sensor in practice for system identification and health monitoring because of its high sensitivity and reliability. Therefore, a local feedback control algorithm with a Kalman filter will be presented in this section for vibration control of the building structure subjected to earthquake excitation using semi-active friction dampers. The primary purpose of implementing local feedback control is to involve minimal feedback signals to achieve reliable and economical control design whereas the use of the Kalman filter makes it possible to use the accelerometers as sensors for both health monitoring and vibration control, leading to the common sensory system and the common data acquisition and
transmission system. The equation of motion of the building with semi-active friction dampers can be expressed as

\[ M_0 \ddot{x} + C_0 \dot{x} + K_0 x = -M_0 \ddot{x}_g + H^T u \]  \hspace{1cm} (6.33)

where \( M_0, C_0, \) and \( K_0 \) are the updated mass, damping, and stiffness matrices of the building, respectively; \( x, \dot{x}, \) and \( \ddot{x} \) are the relative displacement, velocity and acceleration vectors, respectively, with respect to the ground motion; \( \ddot{x}_g \) is the ground acceleration; and \( u \) is the semi-active control force vector; and \( H^T \) is the influence matrix reflecting the location of the semi-active friction dampers.

The semi-active friction damper is modeled with the components of a linear spring and a variable friction damper connected in series, as shown in Figure 6.1 (b). The linear spring simulates the brace connecting the friction damper to the building. The behavior of friction damper is assumed to follow the Coulomb friction model, and the friction force of the damper is therefore linearly proportional to the clamping force when the dynamic friction coefficient \( \mu \) is considered to be constant and independent of sliding velocity and displacement. As a result, the semi-active control force \( u_t \) depends on either sticking or slipping state of the damper and it can be written as

\[
  u_t = \begin{cases} 
    k^d[d_t - e_t] & \text{if } | f_t^k | \leq | f_t^d | \quad \text{(sticking)} \\
    f_t^d & \text{if } | f_t^k | > | f_t^d | \quad \text{(slipping)}
  \end{cases}
\]  \hspace{1cm} (6.34)

\[
f_t^k = k^d[d_t - e_t]
\]  \hspace{1cm} (6.35)

in which \( k^d \) is the spring stiffness (brace stiffness) of the semi-active friction damper; \( f_t^d \) is the friction force of semi-active friction damper; \( f_t^k \) is the working axial force in the semi-active friction damper; \( d_t \) denotes the axial displacement between the two
ends of the friction damper; \(e_i\) is the slip deformation of the friction damper and it is given by

\[
e_i = \bar{e}_i + \left| d_i - \bar{e}_i \right| - \frac{|u_i|}{k_d} \text{sgn}(\dot{d}_i)
\]  

(6.36)

where \(\bar{e}_i\) is the previously cumulated slip deformation of the friction damper and \(\dot{d}_i\) is the relative velocity between the two ends of the friction damper. The friction force of semi-active friction damper is given by

\[
f_i^{d} = \mu N_i^{d} \text{sgn}(\dot{d}_i)
\]  

(6.37)

where \(N_i^{d}\) is the clamping force of semi-active friction dampers which is time dependent and determined by the feedback controller. The semi-active control force \(u_i\) is exactly the same as the semi-active friction force \(f_i^{d}\) if the friction damper slips continuously without sticking. This, however, relies on the control strategy and parameters.

### 6.4.2 Local feedback control strategy

One kind of local feedback control strategy for the semi-active friction damper is the viscous and Reid friction control strategy proposed by Chen et al (2002). In this strategy, the controllable clamping force is the function of displacement and velocity of the semi-active friction damper as given in the following

\[
N_i^{d} = G_g |\dot{d}_i| + G_e |d_i|
\]  

(6.38)

where \(G_e\) and \(G_g\) are the positive gain coefficients for the damper displacement \(d_i\) and the damper velocity \(\dot{d}_i\), respectively. The damper displacement \(d_i\) and the damper velocity \(\dot{d}_i\) are very close to the damper slip \(e_i\) and the damper slip rate \(\dot{e}_i\).
respectively. Thus, by using this strategy the clamping force becomes larger as the damper displacement increases so as to enhance energy dissipation. The clamping force also becomes larger as the damper velocity increases so as to avoid excessive slippage. It is obvious that the viscous and Reid friction control strategy combines both features of viscous damper and non-linear Reid damper, which can be seen by taking either control gain $G_g = 0$ or $G_e = 0$, respectively.

With reference to Figure 6.1 (a) and by assuming one semi-active friction damper for one building storey, the relationship between the $j$th damper displacement (velocity) and the relative displacement (velocity) of the $j$th building storey at time instant $t$ can be expressed as

$$d_j(t) = \cos \theta_j (x_j(t) - x_{j-1}(t))$$  
$$\dot{d}_j(t) = \cos \theta_j (\dot{x}_j(t) - \dot{x}_{j-1}(t))$$  

where $x_j(t)$ and $\dot{x}_j(t)$ are the displacement and velocity of the $j$th building floor; $x_{j-1}(t)$ and $\dot{x}_{j-1}(t)$ are the displacement and velocity of the $(j-1)$th building floor; and $\theta_j$ is the angle between the $j$th semi-active friction damper and the $(j-1)$th building floor. With equations (6.39) and (6.40), the semi-active friction force of the damper, $f_{r,j}^d$, can be determined in terms of the relative displacement and velocity of the building storey.

However, as mentioned before the accelerometer is the mostly-used sensor in practice for system identification and health monitoring because of its high sensitivity and reliability. It will be uneconomical to install many different sensors for health monitoring and response control of one building. Therefore, a local
feedback control strategy with the Kalman filter, by which the acceleration responses of the building are used as feedback signals other than displacement and velocity responses, is presented in this study for vibration control of the building structure subjected to earthquake excitation using semi-active friction dampers. In this regard, the equation of motion of the building with semi-active friction dampers, that is Equation (6.33), is rewritten in state-space form as

\[
\dot{z} = Az + Bu + E\ddot{x}_g
\]  

(6.41)

The measured responses \( y_m \) are selected as the absolute acceleration responses of the building floors.

\[
y_m = C_m z + D_m u + F_m \ddot{x}_g + v
\]  

(6.42)

where

\[
A = \begin{bmatrix} 0 & I \\ -M_0^{-1}K_0 & -M_0^{-1}C_0 \end{bmatrix}; \quad B = \begin{bmatrix} 0 \\ -M_0^{-1}H_c \end{bmatrix}; \quad E = \begin{bmatrix} 0 \\ -1 \end{bmatrix};
\]  

(6.43)

in which \( z = [x \ x]^T \) is the state vector of controlled building; \( u \) is the semi-active control force vector; \( v \) is the measurement noise vector; \( I \) is the unit diagonal matrix; \( C_m, D_m \) and \( F_m \) are the reduced-order coefficient matrices of \( A, B \) and \( E \), respectively.

The estimated state vector \( \hat{z} \) is described by the Kalman filter optimal estimator in the form of

\[
\dot{\hat{z}} = A\hat{z} + Bu + L(y_m - C_m\hat{z} - D_m u)
\]  

(6.44)

\[
L = [R^{-1}(\gamma g F_m E^T + C_m S)]^T
\]  

(6.45)

where \( S \) is the solution of the algebraic Riccati equation given by

\[
SA + A^TS - SGSS + H = 0
\]  

(6.46)
The measurement noises in all the building floor responses are assumed to be the same as a stationary Gaussian white noise process. The power spectral density ratio in Equation (6.45) is defined as $\gamma_{g} = S_{\dot{x}_{g}\dot{v}_{g}} / S_{\dot{v}_{g}}$, where $S_{\dot{x}_{g}\dot{v}_{g}}$ and $S_{\dot{v}_{g}}$ are the power spectral density function of the stationary white noise of $\dot{x}_{g}$ and $\dot{v}_{g}$, respectively.

Once the estimated state vector $\hat{z}$ is obtained from Equation (6.44), the corresponding estimations for displacement and velocity responses, $\hat{x}$ and $\hat{\dot{x}}$, of the building can be obtained. The estimations of damper slip $d_{j}^{l}$ and slip rate $\dot{d}_{j}^{l}$ at time instant $t$ can be then obtained as

$$d_{j}^{l} = (\hat{x}_{j}^{l} - \hat{x}_{j}^{l-1}) \cos \Theta_{j}$$

$$\dot{d}_{j}^{l} = (\hat{\dot{x}}_{j}^{l} - \hat{\dot{x}}_{j}^{l-1}) \cos \Theta_{j}$$

where $\hat{x}_{j}^{l}$ and $\hat{x}_{j}^{l}$ are the estimated displacement and velocity of the $j$th floor of the building; $\hat{x}_{j}^{l-1}$ and $\hat{x}_{j}^{l-1}$ are the estimated displacement and velocity of the $(j-1)$th floor of the building. The estimation of clamping force and the friction force in the semi-active friction damper can be given by

$$N_{t}^{d} = G_{g} | \dot{d}_{j} | + G_{e} | \ddot{d}_{j} |$$

$$f_{t}^{d} = \mu N_{t}^{d} \text{sgn}(\dot{d}_{j})$$

and

$$A = A^{T} - C_{m}^{T}R^{-1}(\gamma_{g}F_{m}E^{T})$$

$$G = C_{m}^{T}R^{-1}C_{m}$$

$$H = \gamma_{g}EE^{T} - \gamma_{g}^{2}EF_{m}^{T}R^{-1}F_{m}E^{T}$$

$$R = I + \gamma_{g}F_{m}F_{m}^{T}$$
Equation (6.34) is then used to determine the semi-active control force, and Equation (6.33) or (6.41) is finally used to determine the structural responses at the next time step. The flow chart of semi-active friction damper using local control strategy with the Kalman filter is plotted in Figure 6.5.

6.4.3 Global feedback control strategy

To provide a comparative basis for the local-feedback control strategy, the linear quadratic Gaussian (LQG) controller with a modified clipped strategy is also applied to the building with semi-active friction dampers. The first two equations of motion of the controlled building in the state-space form using the global feedback control strategy are the same as Equations (6.41) and (6.42) using the local feedback control strategy. The additional equation is given for the regulated response.

$$y_{ed} = C_{ed}z + D_{ed}u + F_{ed}\ddot{x}_g$$  \hspace{1cm} (6.55)

where $y_{ed}$ is the regulated response vector; $C_{ed}$, $D_{ed}$ and $F_{ed}$ are the reduced-order coefficient matrices of $A$, $B$ and $E$, respectively. The acceleration feedback LQG controller is basically designed to minimize a quadratic objective function, in this study, by weighting the absolute acceleration responses of the building and the control forces, which is given by

$$J = \lim_{\tau \to \infty} \frac{1}{\tau} \mathbb{E}\left[\int_{0}^{\tau} \left( (C_{ed}z + D_{ed}u)^T Q (C_{ed}z + D_{ed}u) + u^T R u \right) dt \right]$$  \hspace{1cm} (6.56)

where $Q$ and $R$ are the weighting matrices for acceleration responses and semi-active control forces, respectively. Based on the separation principle that allows feedback gain and Kalman gain determined separately (Stengel 1986; Skelton 1988), the optimal control force vector is obtained as

$$u = -Kz$$  \hspace{1cm} (6.57)
where $K$ is the full state feedback gain matrix given by

$$
K = \tilde{R}^{-1} (\tilde{N}^T + B^T P)
$$

(6.58)

where $P$ is the solution of the algebraic Riccati equation given by

$$
P \tilde{A} + \tilde{A}^T P - P B \tilde{R}^{-1} B^T P + \tilde{Q} = 0
$$

(6.59)

in which

$$
\tilde{Q} = C_{ed}^T Q C_{ed} - \tilde{N} \tilde{R}^{-1} \tilde{N}^T
$$

(6.60)

$$
\tilde{N} = C_{ed}^T Q D_{ed}
$$

(6.61)

$$
\tilde{R} = R + D_{ed}^T Q D_{ed}
$$

(6.62)

$$
\tilde{A} = A - B \tilde{R}^{-1} \tilde{N}^T
$$

(6.63)

Equations (6.41), (6.44) and (6.57) form the basic equations for the active control of the building using the LQG controller. Nevertheless, the control force of a semi-active friction damper depends on its motion status: when the damper is in sticking stage, the control force is equal to the axial force in the brace; and when the damper is slipping stage, the magnitude of control force depends on the controllable clamping force and its direction depends on the velocity of the damper. Therefore, the closed-loop feedback force determined by Equation (6.57) cannot always achieved by the semi-active friction dampers. Furthermore, the clipped-optimal control strategy proposed by Dyke et al (1996) for magnetorheological (MR) damper is also not applicable to the semi-active friction damper because the desired control force could be imitated correctly only when the damper is in slipping state. Switching the clamping force to its maximum level would very likely cause the friction damper stick and the magnitude of sticking force is definitely not consistent with that of the feedback counterpart (Ng and Xu 2004). Therefore, the following modified clipped control strategy is used for the friction-based semi-active devices.
\[ N_i^d = \begin{cases} u_i^{LQG}/\mu & \text{if } \dot{d}_i \cdot u_i^{LQG} \geq 0 \text{ and } |u_i^{LQG}| < \mu N_{\max}^d \\ N_{\max}^d \text{sgn}[u_i^{LQG}] & \text{if } \dot{d}_i \cdot u_i^{LQG} < 0 \text{ and } |u_i^{LQG}| \geq \mu N_{\max}^d \\ 0 & \text{if } \dot{d}_i \cdot u_i^{LQG} < 0 \text{ and } |u_i^{LQG}| < \mu N_{\max}^d \end{cases} \] (6.64)

where \( u_i^{LQG} \) is the optimal active control force determined by the LQG controller; \( N_{\max}^d \) is the maximum clamping force which can be provided by the semi-active friction damper. It can be seen from Equation (6.64) that when the direction of the desired active control force is opposite to the velocity direction of the semi-active friction damper and the magnitude of the desired active control does not exceed the maximum control force which the damper can bear, the semi-active friction damper is able to generate the desired control force in the same direction as required. Since the semi-active friction damper cannot generate the desired active control force in the opposite direction as the velocity direction of the damper, a zero clamping force is hence commanded to produce zero friction force at this time instant. The flow chart of vibration control process using either the local or the global control strategy with the Kalman filter is plotted in Figure 6.6.

### 6.5 Damage detection

Even with control devices installed, building structures may still suffer some damage after extreme events or long-term service. The rational approach is thus necessary to assess the potential damage of a controlled building. In this study, the proposed modal updating scheme is applied to the controlled building with damage to identify the structural parameters of the damaged building based on adding known stiffness using the semi-active friction dampers to create the two states: (1) the damaged building without any additional stiffness (clamping force is set at zero); and (2) the damaged building with additional stiffness (damper is in sticking state). By
comparing with the structural parameters of the undamaged structure, the location and severity of the structural damage can be determined. The eigenvalue problem of the damaged building structure without any additional stiffness (the clamping force is set at zero) can be given by

\[ KX_{0k}^d = MX_{0k}^d \Lambda_{0k}^d \]  \hspace{1cm} (6.65)

where \( K \) and \( M \) are the stiffness and mass matrices of the damaged building structure, respectively; \( X_{0k}^d \) is the measured \( N \times N_e \) modal matrix of the damaged building structure; and \( \Lambda_{0k}^d \) is the \( N_e \times N_e \) diagonal matrix whose elements correspond to the square of the measured natural frequencies of the damaged building structure.

The eigenvalue problem of the damaged building structure with additional stiffness (the damper is in sticking state) can be expressed as

\[ (K + K_a)X_k^d = MX_k^d \Lambda_k^d \]  \hspace{1cm} (6.66)

where \( X_k^d \) is the measured \( N \times N_e \) modal matrix of the damaged building structure with additional stiffness; and \( \Lambda_k^d \) is the \( N_e \times N_e \) diagonal matrix whose elements correspond to the square of the measured natural frequencies of the damaged building structure with additional stiffness.

By following the same procedure as used for the system identification and considering the connectivity information of structural members, the stiffness identification equation for the damaged building can be derived as

\[ \Lambda_k^d \cdot k^d = h_k^d \]  \hspace{1cm} (6.67)

where

\[ k^d = [k_{11}^d, k_{12}^d, \ldots, k_{ij}^d, \ldots, k_{NN}^d]^T \] \hspace{1cm} (abs\((i-j)\) \leq 1)  \hspace{1cm} (6.68)
\[
\mathbf{h}_k^d = [h_{11}^d \cdots h_{N_1}^d | h_{21}^d \cdots h_{N_2}^d | \cdots | h_{N_1,1}^d \cdots h_{N_1,N_2}^d ]^T
\]  

(6.69)

where \( \mathbf{A}_k^d \) is obtained from \( \mathbf{X}_0^d \mathbf{T} \otimes \mathbf{X}_k^d \) by deleting all the columns that multiply by \( k_{ij}^d = 0 \) \((\text{abs}(i - j) > 1)\); the unknown column vector \( \mathbf{k}_k^d \) is of length \((3N - 2)\); and the vector \( \mathbf{h}_k^d \) is of length \(N_e^2\) and its element \( h_{ij}^d \) is equal to \( U_{ij}^d \) in the matrix \( \mathbf{U}_k^d \). The matrix \( \mathbf{U}_k^d \) can be obtained by solving the equation similar to Equation (6.10). For the shear building considered in this study, the relationship between the horizontal storey stiffness coefficients and the elements of the stiffness matrix should be established to obtain the transformation matrix \( \mathbf{T} \). The horizontal storey stiffness of the damaged building, \( \mathbf{k}^p \), can then be obtained by finding the following minimum norm least-squares solution:

\[
\mathbf{k}^p = (\mathbf{A}_k^d \mathbf{T})^T \mathbf{h}_k^d
\]

(6.70)

Once the horizontal storey stiffness coefficients of the damaged building are found, the damage location and severity of the damaged building can be determined by

\[
\Delta \mathbf{k}^p = \mathbf{k}^p - \mathbf{k}_0^p
\]

(6.71)

where \( \mathbf{k}_0^p \) is the horizontal storey stiffness coefficient vector of the original building obtained from the system identification in section 6.3. The flow chart of the proposed damage detection procedure for the building with semi-active friction dampers is plotted in Figure 6.7.

### 6.6 Numerical Study

The proposed integrated procedure can fulfill model updating, seismic response control and damage detection of the building structure in a systematic and interactive way. Nevertheless, the feasibility of the proposed procedure should be investigated.
and some practical issues should be addressed before this procedure can be applied to real building structures.

The model updating scheme in the integrated procedure for identifying the structural parameters of a building is based on the adding known stiffness provided by the semi-active friction dampers. The natural frequencies and mode shapes of the original building with and without additional stiffness should be identified before the structural parameters can be identified. In practice, the natural frequencies and mode shapes are often identified using the measured structural responses to ambient excitations. The noise contamination is unavoidable in the measured structural responses (Zhao et al., 2006). Therefore, the effect of measurement noise on the identification quality of natural frequencies and mode shapes and then structural parameters using the proposed model updating scheme should be examined. Furthermore, it is often difficult in practice, if not impossible, to obtain the same number of natural frequencies and modal shapes as the number of degrees of freedom of the analytical model of the building (Cha and de Pillis 2001). As a result, the measurement information is often incomplete. The effect of incomplete measurement information on the identification quality of structural parameters using the proposed updating scheme should also be examined. Moreover, the feasibility and identification quality of the proposed model updating scheme depends on the adding known stiffness provided by the semi-active friction dampers through braces. It is necessary to perform a parameter study to find optimum damper (brace) stiffness.

For seismic vibration control of the building using the semi-active friction dampers, the local feedback control with a Kalman filter is proposed in the integrated
procedure to make it possible to use accelerometers as common sensors for both health monitoring and vibration control. The control performance of such a local feedback control strategy should be evaluated and compared with those using the same local feedback control strategy but without the Kalman filter and using the global feedback control strategy. The optimum parameters involved in both the control strategy and control device should be determined and the robustness of the control system should be examined through parameter studies considering various earthquake inputs. The feasibility of the proposed integrated procedure also depends on the damper (brace) stiffness for both vibration control and model updating, which deserve a further parameter study to find its optimum value for both purposes.

As for the damage detection scheme in the integrated procedure, the feasibility of this scheme for identifying different damage severities and locations should be examined. Since this scheme is closely related to the model updating scheme, the effect of noise contamination on the damage detection quality should be investigated. This scheme should also be compared with the traditional sensitivity approach to ascertain its damage detection quality. The preceding tasks in each of the three components, concerning the feasibility of the proposed integrated procedure and the practical issues of its application, will be performed in this section through the numerical analyses of an example shear building.

6.6.1 Parameter identification

6.6.1.1 Description of an example building

A simple five-storey shear building, as shown in Figure 6.8, is selected as an example building to facilitate our discussions on the concept and quality of the
The proposed integrated procedure. The example building has the same storey height of 3 \( m \). The mass and the horizontal storey (shear) stiffness of the original building without damage are uniform for all storeys with mass \( m = 5.1 \times 10^3 \) kg and stiffness \( k = 1.334 \times 10^7 \) N/m. The Rayleigh damping assumption is adopted to construct the structural damping matrix. The damping ratios in the first two modes of vibration of the building are assumed to be 0.02. A semi-active friction damper with a diagonal brace is used to connect two neighboring floors to provide additional stiffness for parameter identification and damage detection and to provide control force for seismic response mitigation. The same arrangement with the same damper is made for each storey of the building. The five accelerometers are installed on the building, each for one floor, to measure the acceleration responses of the building subjected to either ambient ground excitation in the case of model updating and damage detection or earthquake ground excitation in the case of vibration control. In the case of model updating, the clamping forces of the semi-active friction dampers are set at zero level first to create the status of the original building without any additional stiffness. The clamping forces of the semi-active friction dampers are then set to the maximum value to ensure all the semi-active friction dampers are in sticking state when the building is subjected to ambient ground excitation so as to create the status of the original building with additional known stiffness. The stiffness ratio (SR) of the additional stiffness from the brace to the horizontal stiffness of the building storey is defined as

\[
SR = \frac{K_d}{K_s}
\]  \hspace{1cm} (6.72)

where \( K_s \) is the horizontal stiffness of the building storey and \( K_d \) is the additional stiffness from the brace when the semi-active friction damper is in sticking state.
6.6.1.2 Parameter identification without noise contamination

As mentioned before, the model updating by adding known mass suggested by Cha and de Pillis (2001) is not a favorable scheme for most of civil engineering structures subjected to earthquake excitation. The identification of structural parameters first and the construction of stiffness and mass matrices afterwards using the proposed model updating scheme are more feasible. This section will take the example building without considering noise contamination to discuss this issue in detail.

Since noise contamination is not considered, the natural frequencies and mode shapes of the five-storey shear building concerned in this study can be obtained through eigenvalue analysis as the exact values for the parameter identification. The natural frequency matrix $\Lambda_{0k}$ and the mode shape matrix $X_{0k}$ of the original building without additional stiffness (the clamping forces of all the semi-active friction dampers are set zero) are computed, and the five natural frequencies are obtained as 2.317, 6.762, 10.661, 13.695, and 15.620 Hz. The stiffness ratios ($SR$) of all the five semi-active friction dampers are then set to be the same of 0.9. The natural frequency matrix $\Lambda_k$ and the mode shape matrix $X_k$ of the building with the additional stiffness (all the dampers are in sticking state) are computed, and the five natural frequencies become 3.193, 9.322, 14.695, 18.877, and 21.530 Hz. For the five-storey shear building concerned, the horizontal storey stiffness coefficients to be identified are 5:

$$k_0^p = [k_1, k_2, k_3, k_4, k_5]^T$$ (6.73)

The stiffness coefficients in the stiffness matrix to be identified are 13 if the sparsity information is taken into consideration:

$$k_0^m = [k_{11}, k_{12}, k_{21}, k_{22}, k_{23}, k_{32}, k_{33}, k_{34}, k_{43}, k_{44}, k_{45}, k_{54}, k_{55}]^T$$ (6.74)

The transformation matrix $T$ between $k_0^p$ and $k_0^m$ can be given by
Let us identify the stiffness matrix first using Equation (6.16). As discussed in before, the number of modes of vibration of the building, $N_e$, used for the identification of stiffness matrix should meet the following condition if one wants to avoid the underdetermined identification problem:

$$N_e \geq \sqrt{3N - 2} \quad (6.76)$$

where $N$ is the number of degrees of freedom of the building. For the five-storey shear building concerned in this study, the number of modes of vibration should be at least 4 if one wants to avoid the underdetermined identification problem. The identified results in terms of the relative identification error are listed in Table 6.1 for $N_e = 2, 3, \text{ and } 4$. The relative identification error in Table 6.1 is defined as the absolute value of the difference between the identified stiffness coefficient and the actual stiffness coefficient divided by the actual stiffness coefficient. It can be seen that if the number of modes of vibration used is less than 4, the relative identification errors are considerable and the symmetric property of the stiffness matrix cannot be maintained. When the number of modes of vibration used is 4, the exact stiffness matrix can be identified.

Let us now identify the horizontal storey stiffness coefficients directly using Equation (6.21). The number of modes of vibration of the building, $N_e$, used for the identification of horizontal storey stiffness coefficients should meet the following condition if one wants to avoid the underdetermined identification problem:
\[ N_e \geq \sqrt{N} \quad (6.77) \]

For the five-storey shear building concerned in this study, the number of modes of vibration should be 3 if one wants to avoid the underdetermined identification problem. The identified results in terms of the relative identification errors are listed in Table 6.2 for \( N_e = 2, 3, \) and 4. It can be seen that only using 3 modes of vibration can give the exact values of five horizontal storey stiffness coefficients and then the exact values of the stiffness matrix. The similar procedure can be used to identify the mass of the each floor and then the mass matrix. By using the first 3 modes of vibration without considering noise contamination, the exact mass coefficients of the building can be identified and the identified results are shown in Figure 6.9. Because the ratio of \( \sqrt{N} / \sqrt{3N - 2} \) approaches 0.58 when \( N \) tends to be infinite, the number of modes of vibration required for the identification of stiffness matrix using the proposed model updating scheme is 40% less than that required for the direct identification of stiffness matrix as far as the shear building is concerned.

**6.6.1.3 Effects of noise contamination**

In practice, the natural frequencies and mode shapes of a building are often identified using the measured acceleration responses with some identification techniques. The noise contamination is unavoidable in the measured acceleration responses and the identification technique may also introduce some errors in the identification of natural frequencies and mode shapes (Zhao et al., 2006). Therefore, the effects of measurement noise and identification technique on the identification quality of natural frequencies and mode shapes and then structural parameters using the proposed model updating scheme should be examined. In this regard, the time history of ground motion is simulated as an input to the building. The ground motion
is assumed to be a white noise random process with a peak value of 0.05 m/s$^2$. The sampling frequency of 500 Hz and the time duration of 200 seconds are used in the simulation. The acceleration responses of all the building floors to the input ground motion are then computed. The measurement noise, which is also assumed to be a white noise random process, is simulated and added to the acceleration response according to a given noise intensity. The contaminated acceleration responses are finally used to identify the natural frequencies and mode shapes and to assess the effects on the identification quality of structural parameters. The noise intensity is defined as the ratio of the root mean square (RMS) of the noise to the RMS of the acceleration response.

$$\text{Noise intensity} = \frac{\text{RMS(noise)}}{\text{RMS(acceleration response)}} \times 100\% \quad (6.78)$$

Two types of modal identification techniques are adopted to identify the natural frequencies and mode shapes: one is based on the known ground motion, and the other is based on the unknown ground motion. For the technique using the known ground motion, the frequency response functions (FRF) of all the building floors are first computed to determine impulse response functions (Ewins 2000). The Eigensystem Realization Algorithm (ERA) is then applied to the impulse response functions to obtain the modal information (Juang 1994). For the technique using the unknown ground motion, the Natural Excitation Technique (NExT) is applied to the acceleration responses to obtain the cross-correlation functions which have the same form as the impulse response functions (James et al., 1995). The ERA is then applied to the cross-correlation functions to obtain the modal information (Farrar and James 1997).
The obtained modal information based on the unknown ground motion is compared with that based on the known ground motion in terms of relative identification error to see the effect of the unknown ground motion on the identification quality of modal information. The relative identification error in the $i$th natural frequency is defined as the absolute value of the difference between the identified $i$th natural frequency and the actual $i$th natural frequency and then divided by the actual $i$th natural frequency. Similarly, the relative identification error in the $r$th component of the $i$th mode shape is defined as the absolute value of the difference between the identified $r$th component of the $i$th mode shape and the actual $r$th component of the $i$th mode shape divided by the actual $r$th component of the $i$th mode shape. Figure 6.10 shows the relative identification errors in the five natural frequencies for the four cases: (1) with the known ground motion and without noise, (2) with the known ground motion and with 1% noise intensity, (3) without knowing the ground motion and without noise, and (4) without knowing the ground motion and with 1% noise intensity. It can be seen that for any one of the four cases, the relative identification error is greater in higher natural frequency. With the known ground motion, the relative identification error in any one of the five natural frequencies is less than 0.4% even with 1% noise intensity. If the ground motion is unknown, the relative identification errors in the first three natural frequencies are less than 1.2% without noise and less than 2% with 1% noise intensity. However, the relative identification error in the fifth natural frequency reaches more than 4% when the ground motion is unknown and the noise intensity is 1%.

Figure 6.11 shows the relative identification errors in the five mode shapes obtained using the known ground motion without noise and with 1% noise intensity. It can be
seen that even with 1% noise intensity, the maximum relative identification error in the five mode shapes is less than 1%. Figure 6.12 depicts the relative identification errors in the five mode shapes obtained based on the unknown ground motion without noise and with 1% noise intensity. The comparison of the results shown in Figure 6.12 with those shown in Figure 6.11 clearly demonstrates that the identification errors due to the unknown ground motion are considerably larger than those with the known ground motion. The maximum relative identification error increases from the first mode shape to the fifth modal shape. The maximum identification error in the fifth modal shape with 1% noise intensity is more than 15%. Nevertheless, the maximum identification error in the first three mode shapes is less than 5% even with 1% noise intensity. These results indicate that when the ground motion is unknown, it will be better to use the lower identified mode shapes for the identification of structural parameters.

The first three natural frequencies and mode shapes of the example building with and without the additional stiffness, identified based on the known ground motion and the unknown ground motion with 1% noise intensity, are then used to identify the horizontal storey stiffness coefficients and mass coefficients of the example building in terms of the proposed model updating scheme. The stiffness ratio referring to the additional stiffness is 0.9 for all the storeys. The relative identification errors in the stiffness coefficients and mass coefficients are depicted in Figures 6.13 (a) and 6.13 (b) for the case of the known ground motion and the unknown ground motion, respectively. It can be seen that the maximum identification error in the stiffness and mass coefficients is about 1% only when the first three natural frequencies and mode shapes of the building identified using the known ground motion are used. When the
first three natural frequencies and mode shapes of the building identified without knowing the ground motion are used, the maximum identification error in the mass and stiffness coefficients could reach about 5%. The identification results are very good in the case of knowing the ground motion and they are also satisfactory in the case without knowing the ground motion.

Two more noise intensities, 0.5% and 2%, are introduced to the acceleration responses of the building to obtain the two more sets of contaminated modal information to assess the effects of noise contamination on the identification quality of the structural parameters. Plotted in Figures 6.14 and 6.15 are the relative identification errors in the stiffness coefficients and mass coefficients using the first three natural frequencies and modal shapes identified with the known and the unknown ground motion, respectively, at the three levels of noise. It can be seen from Figure 6.14 that when the first three natural frequencies and mode shapes identified using the known ground motion are used in the identification of structural parameters, the maximum relative identification error in the stiffness and mass coefficients is less than 3% even with 2% noise intensity. The proposed model updating scheme is very effective. When the first three natural frequencies and mode shapes identified without knowing the ground motion are used, the maximum identification error can reach about 7% in the mass coefficients and 9% in the stiffness coefficients.

6.6.1.4 Effects of higher modal information
As discussed before, the use of the first three modes of vibration can identify exactly the actual structural parameters of the example building if the actual natural
The relative identification errors in the stiffness coefficients and in the mass coefficients are showed in Figures 6.16 (a) and 6.16 (b), respectively, with the first four modes of vibration included. The same quantities with all the five modes of vibration included are depicted in Figures 6.16 (c) and 6.16 (d). Compared with the results shown in Figures 6.15(a) and 6.15 (b) with only the first three modes of vibration included, one may realize that the inclusion of higher modes of vibration in the identification of structural parameters could not improve the identification quality. On the contrary, it deteriorates the identification quality, in particular in the identification of stiffness coefficients. This is because the identification errors in the fourth and fifth modal information without knowing the ground motion are much higher than those in the first, second and third modal information, as shown in Figures 6.10 and 6.12. Furthermore, the identification equation for the structural parameters (see Equation (6.21)) indicates that for the concerned five-storey shear building, the use of the first three modes of vibration can avoid the underdetermined identification problem already. The further inclusion of the fourth and fifth modes of vibration is not necessary, as highlighted by the results tabulated in Table 6.2. The effect of higher modes of vibration on the identification quality of the structural parameters using the natural frequencies and mode shapes identified from the contaminated acceleration responses is investigated in this section. Only the cases of the natural frequencies and mode shapes identified without knowing the ground motion are considered in this section at different noise levels for the problem concerned. The stiffness ratio remains unchanged as 0.9 for all the storeys.
vibration leads to the overdetermined identification problem. Therefore, the minimum number of modes of vibration to avoid the underdetermined identification problem is the best number for the identification when using the proposed model updating scheme from a practical point of view. Of course, if the information on input excitation can be obtained and the natural frequencies and mode shapes are identified using the known input excitation the identification quality of the structural parameters using the proposed scheme could be enhanced significantly.

6.6.1.5 Effects of additional stiffness

It can be understood from the proposed model updating scheme that the identification process actually involves the modal information of the building without and with additional stiffness. The change in additional stiffness will lead to the change in natural frequencies and mode shapes, which will in turn affect the identification of structural parameters. Therefore, the effect of additional stiffness on the identification quality of structural parameters should be investigated. The relative changes in the five natural frequencies with the stiffness ratio (SR) for the example building are computed and shown in Figure 6.17. The relative change in the $i$th natural frequency is defined as the difference between the $i$th natural frequencies of the building with and without additional stiffness divided by the $i$th natural frequency of the building without additional stiffness. The value of SR used is the same for all the storeys. It can be seen from Figure 6.17 that the relative changes in natural frequencies increase with the increase of SR value. The relative changes in the five natural frequencies for a given SR value, however, remain the same.
Clearly, smaller additional stiffness yields smaller relative change in modal properties. The identification quality of smaller change in modal properties will then be easily affected by the measurement noise and the modal identification technique without knowing the excitation. On the other hand, too large additional stiffness may not be feasible or economic in practice. Therefore, there exists a minimum SR value for the brace used together with semi-active friction dampers to ensure satisfactory identification quality. Before performing a numerical investigation on this issue, the average relative identification error for the five stiffness coefficients is defined as the average values of the five relative identification errors in the five stiffness coefficients. The computed results of the average identification errors against SR are plotted in Figure 6.18 at the three levels of noise intensity. It can be seen that larger identification errors occur when SR values are smaller. This is because the smaller change in modal properties is overlapped by the identification errors due to measurement noise and unknown ground motion. With the increase of SR values, the average identification errors reduce rapidly first and then approach to constant values. For the concerned example building, the SR value of 0.9 can be selected as a proper value for the additional stiffness. Nevertheless, the final value of SR selected also depends on vibration control performance because the value of SR also affects the control performance of semi-active friction dampers for seismic response control. This issue will be discussed in the subsequent section.

6.6.2 Seismic response control

6.6.2.1 Seismic inputs and structural parameters

To evaluate the control performance and robustness of the semi-active friction dampers, four seismic records are selected as inputs to the example building: (1) El
Centro NS (1940); (2) Hachinohe NS (1968); (3) Northridge NS (1994); and (4) Kobe NS (1995). The time histories of the four seismic records are shown in Figure 6.19. The original peak ground accelerations (PGA) of the four seismic records are 3.417, 2.250, 8.267, and 8.178 m/s², respectively. The original time histories of the four seismic records are scaled to have the same PGA of 4.0 m/s² to facilitate the comparison. In consideration of the problem from a practical viewpoint, the stiffness and mass matrices of the example building are constructed using the structural parameters identified in section 2 with 1% noise intensity but without knowing input. The damping matrix is constructed based on the Rayleigh damping assumption and the first two modal damping ratios of 1.7% identified in section 2 with 1% noise intensity but without knowing input. The stiffness ratios of all the five semi-active friction dampers are kept the same value of 0.9 unless otherwise specified.

6.6.2.2 Control strategies and evaluation index

For seismic response control of a building using semi-active friction dampers, a local feedback control strategy with a Kalman filter is proposed in this chapter in order to make it possible to use accelerometers as common sensors for both health monitoring and vibration control. The control performance of the local feedback control strategy with the Kalman filter is evaluated through comparisons with those using the same local feedback control strategy but without the Kalman filter and with the global feedback control algorithm. As a result, three control strategies are used for performance evaluation: (1) local feedback control without a Kalman Filter; (2) local feedback control with a Kalman Filter; and (3) linear quadratic Gaussian (LQG) global feedback control.
To implement the first control strategy in practice, five displacement sensors and five velocity sensors with one displacement sensor and one velocity sensor for one building floor are required to obtain the feedback. To implement the second control strategy in practice, only five accelerometers with one for one building floor are needed to obtain the feedback. For the implementation of the third control strategy in practice, five accelerometers and five force transducers with one accelerometer and one force transducer for one building storey are necessary to realize the feedback control. Clearly, the second control strategy is most attractive in terms of the type and number of sensors required if the control performance using this strategy is compatible with other two control strategies. In the numerical investigation of control performance in this section, the corresponding computed building responses and damper forces are taken as the relevant feedback instead of the signals from the sensors in practice. The control performance is evaluated in terms of a vibration reduction factor ($VRF$) defined as follows (Xu et al., 2001):

$$VRF = \frac{Z_{nc} - Z_{co}}{Z_{nc}}$$

(6.79)

where $Z_{nc}$ is the maximum response (either displacement, velocity, or acceleration) of a given building floor without control; and $Z_{co}$ is the maximum response of the same quantity of the same floor with control.

6.6.2.3 Optimum gain coefficient for local control strategy

For the local control strategy, the controllable clamping force is the function of displacement and velocity of the semi-active friction damper through two gain coefficients $G_c$ and $G_g$, as shown in Equation (6.38). Chen and Chen (2002) further suggested taking a gain ratio $G_c/G_g = 2\omega_k/\pi$ for the local feedback controller, in
which $\omega_1$ denotes the first natural frequency of the uncontrolled building. Therefore, only the optimum gain coefficient $G_e$ should be determined in this study. For the local control strategy with the Kalman filter, the measurement noise intensity is taken as 1%, which is the same as that used in the system identification, for all the building floor responses. The power spectral density ratio $\gamma_g$, as shown in Equation (6.45), is calculated as 13.12 for the El Centro NS seismic input and for 1% noise intensity. The friction coefficient is taken as 0.2 for all the semi-active friction dampers. The vibration reduction factor ($VRF$) of the example building using the local control strategy with the Kalman filter is then computed for a series of gain coefficient $G_g$.

The mean value of the vibration reduction factors for all the building floors is plotted in Figure 6.20 against the gain coefficient $G_e$ for the displacement, velocity and acceleration response of the building, respectively. It can be seen from Figure 6.20 that as the gain coefficient increases, the mean vibration reduction factor of either displacement or velocity response increases. However, the mean vibration reduction factor of acceleration response increases as the gain coefficient increases up to a value about $0.8 \times 10^7$, but it decreases with the further increase of the gain coefficient. This is because the large gain coefficient may amplify the clamping force and slipping force of the semi-active friction damper which may in turn make the damper easy to stick and increase acceleration responses accordingly. Based on the proceeding discussion, the optimum gain coefficient for the local control strategy with the Kalman filter is taken as $0.8 \times 10^7$. By using this value, satisfactory control performance can be achieved for all the three kinds of seismic responses of the example building using the local control strategies.


**6.6.2.4 Comparison of three control strategies**

In the implementation of the global feedback control strategy, the two weighting matrices $Q$ and $R$ are selected as the unit diagonal matrix multiplied by a factor. The optimum factor is found to be $2.1 \times 10^5$ for $Q$ and $0.016$ for $R$. To determine the maximum clamping force $N_{\text{max}}^d$, which can be provided by the semi-active friction damper and is shown in Equation (6.64), the maximum axial force in the brace of the exampled building without semi-active control is computed with the El Centro NS seismic input. The maximum axial force is then taken as the maximum slipping force.

Figures 6.21 (a)-(c) depict the variation of the peak displacement, velocity, and acceleration responses of the example building with the building floor under the El Centro NS seismic ground motion for five cases: (1) original building without any control; (2) original building with braces; (3) local control without Kalman filter; (4) local control with Kalman filter; and (5) global feedback control. It can be seen that the installation of common braces can reduce the maximum displacement response of the building but it can increase the maximum acceleration response compared with the original building. With any one of the three control strategies, the maximum responses of displacement, velocity and acceleration of the building are all reduced compared with the original building. The control performances of the two local feedback strategies are very close to each other. The semi-active friction dampers with the global control strategy demonstrate the best control performance in the sense that it mostly reduces all the three kinds of seismic responses at all the building floors. In consideration of relatively simple sensor system in the local control strategy with the Kalman filter, the performance of this control strategy is satisfactory.
Demonstrated in Figures 6.22 (a)-(c) are the displacement, velocity and acceleration time histories of the top floor predicted using the local control strategy with the Kalman filter. The actual displacement, velocity and acceleration time histories of the top building floor used for the local control strategy without the Kalman filter are also illustrated in Figures 6.22 (a)-(c). The comparison between the two sets of time histories clearly reveals that the Kalman filter can be used to effectively estimate the displacement and velocity responses from the acceleration responses within the whole control period. This also indicates the accurate estimation of damper slipping states and control force by using acceleration responses only. Therefore, the complexity and expense of the sensory system for seismic response control of the building can be improved to some extent by using the proposed local control strategy with the Kalman filter. Displayed in Figures 6.23 (a) to (c) are the time histories of displacement, velocity, and acceleration responses at the top floor of the building without any control and with the local control strategy together with the Kalman filter. Clearly, the local control strategy with the Kalman filter can effectively suppress the seismic responses of the building when the building experiences large vibration.

6.6.2.5 Effects of brace stiffness

The feasibility of the proposed integrated procedure also depends on the brace (damper) stiffness for both vibration control and model updating. Figure 6.24 displays the variation of $VRF$ of displacement, velocity and acceleration responses of the top floor with $SR$ under the El Centro NS earthquake by using the local control strategy with the Kalman filter. It can be seen that the vibration reduction factors for all the three responses increase rapidly when the stiffness ratio increases from 0.0 to
Afterwards, the vibration reduction factors increase only slightly with increasing stiffness ratio. When it reaches above 0.9, the stiffness ratio has almost no effect on the vibration reduction factors. The similar results are found for other building floors. As a result, the optimum stiffness ratio for seismic response control of the example building should be 0.9, which is consistent with the optimum stiffness ratio determined for system identification of the same building. Nevertheless, almost the same optimum stiffness ratio for both seismic response control and system identification of the example building may not be true for other buildings.

6.6.2.6 Control performance under other seismic inputs

All the above observations are made for the example building under the El Centro NS earthquake. The similar investigations are performed on the example building with other three seismic inputs. It is found that the optimum stiffness ratio found from the case of the El Centro NS earthquake remains almost unchanged for the other three seismic inputs. The optimum gain coefficient obtained from the case of the El Centro NS earthquake can also be applied to the cases of other three seismic inputs although there are slightly different values for different seismic inputs. To have a reasonable comparison of control performance of the semi-active friction dampers manipulated by three control strategies for the example building under different seismic inputs, two sets of normalized performance indices are used. The first set of the performance indices is related to the building responses (Spencer et al., 1998; Ohtori et al., 2004). They include peak- and RMS- based interstory drift ratios \(J_1\) and \(J_3\) and peak- and RMS-based absolute acceleration responses \(J_2\) and \(J_4\).

\[
J_1 = \left\{ \frac{\max_{t \in \mathbb{R}} |dx_i(t)|/h_i}{\delta_{\text{max}}} \right\} 
\]

(6.80)
\[ J_2 = \left\{ \frac{\max_{i,j} \mid \ddot{x}_{ii}(t) \mid}{\dot{x}_{ii}^\text{max}} \right\} \]  \hspace{1cm} (6.81)

\[ J_3 = \left\{ \frac{\max_{i,j} \| d\chi(t) \| / h_i}{\| \delta^\text{max} \|} \right\} \]  \hspace{1cm} (6.82)

\[ J_4 = \left\{ \frac{\max_{i,j} \| \ddot{x}_{ii}(t) \|}{\| \dot{x}_{ii}^\text{max} \|} \right\} \]  \hspace{1cm} (6.83)

where \( d\chi(t) \) is the interstory drift of the \( i \)th storey of the building with control; \( h_i \) is the height of the \( i \)th storey; \( d\chi(t)/h_i \) is the interstory drift ratio of the \( i \)th storey of the building with control; \( \delta^\text{max} \) is the maximum interstory drift ratio of the original building without any control; \( \ddot{x}_{ii} \) is the absolute acceleration response of the \( i \)th floor of the building with control; \( \ddot{x}_{ii}^\text{max} \) is the maximum absolute acceleration response of the \( i \)th floor of the building without any control. The RMS response quantities within the time duration \( t_f \) under each earthquake are calculated by

\[ \| \cdot \| = \sqrt{\frac{1}{t_f} \int_0^{t_f} [\cdot]^2 \, dt} \]

The sign \( \max_{i,j} \) means to find the maximum value within the given time duration first and among all the building storeys/floors afterwards.

The second set of performance indices are related to the capacity of control devices. Only the peak-based control force \( (J_5) \) is used in this study.

\[ J_5 = \left\{ \frac{\max_{i,j} \mid f_i(t) \mid}{W} \right\} \]  \hspace{1cm} (6.84)

where \( f_i(t) \) is the control force generated by the \( i \)th control device; and \( W \) is the seismic weight of the building, that is, the total weight of all the building floors in this study.
The performance indices of the example building with the semi-active friction dampers manipulated by the three control strategies are respectively computed using Equations (6.80) to (6.84) for each of four seismic inputs. Listed in Table 6.3 and Table 6.4 are the performance indices of the controlled building using the local control strategy without and with the Kalman filter, respectively. The performance indices of the controlled building using the global control strategy are tabulated in Table 6.5. It can be seen that the three control strategies all can effectively reduce the RMS responses of the example, with the global control strategy being the most effective one. The peak responses of the example building can also be reduced but they are not as effective as the RMS responses, in particular in the cases of Hachinohe and Northridge earthquakes. The control forces when using the global control strategy are however much larger than those when using the local control strategies. It can also be seen that the control performance of the two local control strategies and the control forces required are similar to each other. These observations together with the sensory system required for each control strategy clearly demonstrate that the proposed local control strategy with the Kalman filter is superior to the local control strategy without the Kalman filter for the integrated health monitoring and vibration control system.

6.6.3 Damage detection

6.6.3.1 Damage scenarios and damage detection

The feasibility of the damage detection scheme in the proposed integrated procedure for identifying different damage severities and locations is examined in this section. The effect of noise contamination on the damage detection quality is also assessed. Two damage scenarios are taken into consideration in this section: (1) single damage
event with 20%, 30%, 40% and 50% stiffness loss, respectively, at the first storey of
the example building; and (2) double damage event with 20% and 30% stiffness loss
at the first and fourth storey respectively. The two levels of measurement noise of
1% and 2% are considered in the damage detection compared with the case without
measurement noise. In the case of damage detection, the clamping forces of the semi-
active friction dampers are set at zero level first to create the status of the damaged
building without any additional stiffness. The clamping forces of the semi-active
friction dampers are then set to the maximum value to ensure all the semi-active
friction dampers are in sticking state when the damaged building is subjected to
ambient ground motion so as to create the status of the damaged building with
additional known stiffness. The ambient ground motion used is the same as that used
in the model updating. The acceleration responses of all the building floors to the
input ground motion are computed for the damaged building with and without
additional stiffness. The measurement noise is simulated and added to the
acceleration response according to a given noise intensity. The contaminated
acceleration responses are then used to identify the natural frequencies and mode
shapes, to detect the damage severity and location, and to assess the effects on the
quality of damage detection.

The damage size, which is defined as the absolute value of the difference between
the detected stiffness and the original stiffness divided by the original stiffness, is
computed for the single damage event, and the results are displayed in Figures 6.25
(a) to (d) for 20%, 30%, 40% and 50% stiffness loss cases, respectively. It can be
seen that without measurement noise, the proposed detection scheme can accurately
determine the damage location and size by using the first three natural frequencies
and mode shapes only. With measurement noise involved, the proposed detection scheme can still determine the damage location and size satisfactorily. Of course, with the increase of measurement noise level, the damage size detected is less accurate. Displayed in Figure 6.26 are the damage sizes for the double damage event. It can be seen that without measurement noise, the proposed detection scheme can exactly determine the damage location and size. With measurement noise considered, the damage location and size can still be effectively determined but the accuracy of damage detection is reduced with the increase of measurement noise level.

6.6.3.2 Comparison with sensitivity based approach

The quality of damage detection using the proposed scheme is compared with that using the traditional sensitivity-based approach. By using the principle of structural sensitivity analysis (Friswell 1995), the equation for the first order sensitivity of eigenvalue and eigenvector to the structural parameter $p_j$ can be expressed in terms of the mass orthogonal conditions $\phi_j^T M \phi_j = 1$ as follows:

$$ \frac{\partial \lambda_i}{\partial p_j} = \phi_i^T \left( \frac{\partial K}{\partial p_j} - \lambda_i \frac{\partial M}{\partial p_j} \right) \phi_j $$

$$ \left( 6.85 \right) $$

$$ \frac{\partial \phi_i}{\partial p_j} = \sum_{k=1,k \neq i}^{n} \phi_i^T \left( \frac{\partial K}{\partial p_j} - \lambda_i \frac{\partial M}{\partial p_j} \right) \phi_k - \frac{1}{2} \phi_i^T \frac{\partial M}{\partial p_j} \phi_i \phi_i $$

$$ \left( 6.86 \right) $$

where $\lambda_i$ and $\phi_i$ are the $i$th eigenvalue and eigenvector of the structure, respectively.

The incremental equation can then be written as

$$ \delta z = S \delta p $$

$$ \left( 6.87 \right) $$

where $S$ is the sensitivity matrix; $\delta z$ is the change in eigenvalue and eigenvector; and $\delta p$ is the change in structural parameters.
The least squares solution for the damage detection equation is

\[ \delta \mathbf{p} = \mathbf{S}^* \delta \mathbf{z} \]  

where \( \mathbf{S}^* \) is the pseudoinverse of the sensitivity matrix \( \mathbf{S} \). The solution for Equation (6.91) begins with an initial estimation of structural parameter \( \mathbf{p}_0 \) to obtain the sensitivity matrix \( \mathbf{S}_0 \) and vector \( \delta \mathbf{z}_0 \). An iterative process is then carried out to improve the accuracy of solution. The least squares solution at the \( k \)th step can be written as

\[ \delta \mathbf{p}_k = \mathbf{S}_k^* \delta \mathbf{z}_k \]  

\[ \mathbf{p}_{k+1} = \mathbf{p}_k + \delta \mathbf{p}_k \]  

The convergence condition for the solution is

\[ \frac{\| \mathbf{p}_{k+1} - \mathbf{p}_k \|}{\| \mathbf{p}_k \|} \leq \varepsilon \]  

where \( \| \cdot \| \) is the norm of vector; \( \varepsilon \) is the prescribed convergence value which is 0.001 in this study.

The sensitivity-based approach as described above is applied to the second scenario of the example building without and with the same measurement noise as used in the
propose a new approach for detecting damages. The damage size detected using the sensitivity-base
approach for each storey is plotted in Figure 6.27. The number of involved natural
frequencies and mode shapes is three. It can be seen that without measurement noise,
both the damage location and size can be accurately predicted by the sensitivity-
based approach. With the measurement noise, the damage location and size can still
be effectively determined but the accuracy of damage detection is reduced with the
increase of measurement noise level. By comparing the results in Figure 6.27 with
the results in Figure 6.26, one may see that the quality of damage detection using the
proposed scheme is similar to that using the sensitivity-based approach. Nevertheless,
the sensitivity-based approach requires that the generalized mass or the mass matrix
is known a priori. The proposed detection scheme together with the proposed
identification scheme, however, does not need such a requirement.

6.7 SUMMARY

An integrated procedure for vibration control and health monitoring of a building
structure using semi-active friction dampers has been proposed in this chapter. For
system identification, a model updating scheme based on adding known stiffness
using the semi-active friction dampers and considering structural connectivity and
transformation information has been proposed for the identification of the structural
parameters and updating the stiffness and mass matrices of the building. For
vibration control, a local feedback control strategy with a Kalman filter has been
proposed for the semi-active friction dampers to reduce seismic responses of a shear
building by using the same accelerometers as used in the system identification. For
damage detection, a damage detection scheme by applying the proposed system
identification method to both the original building and damaged building has been
suggested. The feasibility of the integrated procedure for model updating, vibration control and damage detection of a building structure using semi-active friction dampers and the practical issues of its application have been investigated numerically.

The numerical results from model updating and system identification clearly demonstrate that the identification of structural parameters first and the construction of stiffness and mass matrices afterwards using the proposed model updating scheme are more feasible and accurate than the existing method of directly updating the stiffness and mass matrices. The minimum number of modes of vibration for avoiding the underdetermined identification problem is the best number for the identification when using the proposed model updating scheme. The further inclusion of higher modes of vibration is not necessary. If the information on input excitation can be obtained and the natural frequencies and mode shapes are identified using the known input excitation, the identification quality of the structural parameters using the proposed scheme could be enhanced significantly. The numerical results from seismic response control demonstrate that the control performance of the local control strategy with the Kalman filter is similar to that using the local control strategy without the Kalman filter. Because the sensory system required by the local control strategy with the Kalman filter is accelerometers only, it is thus superior to the local control strategy without the Kalman filter for the integrated health monitoring and vibration control. It is also found that for the example building, the optimum stiffness ratio for seismic response control is almost the same as that for system identification. The numerical results from damage detection demonstrate that the quality of damage detection using the proposed scheme is similar to that using the traditional sensitivity-based approach. The sensitivity-based approach requires
that the generalized mass or the mass matrix is known a priori, but the proposed
detection scheme together with the proposed identification scheme does not need
such a requirement.

The example shear building is adopted in this chapter to examine the feasibility of
proposed integrated vibration control and health monitoring system using semi-active
friction dampers. Many practical factors are taken into consideration which include
the effects of measurement noise and incomplete measurement information on the
system identification and damage detection, the selection of proper brace stiffness for
both vibration control and health monitoring, the effectiveness of the proposed local
feedback control algorithm compared with the global feedback control algorithm;
and the feasibility of detecting both damage location and severity compared with the
traditional sensitivity-based damage detection method. Based on the observations
made from the shear building in this chapter, the integrated monitoring and control
system will be applied to the reticulated shell in the next chapter. Because the
structural configuration of the reticulated shell is more complicated than the simple
shear building, the realization of vibration control, model updating and damage
detection using semi-active friction dampers for the reticulated shell will be different
to some extent.
Table 6.1 Relative identification errors in the direct identification of stiffness matrix elements

<table>
<thead>
<tr>
<th>Number</th>
<th>Elements</th>
<th>Identification Errors (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Ne=2</td>
</tr>
<tr>
<td>1</td>
<td>k_{11}</td>
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<td>2</td>
<td>k_{12}</td>
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<tr>
<td>3</td>
<td>k_{21}</td>
<td>274.69</td>
</tr>
<tr>
<td>4</td>
<td>k_{22}</td>
<td>97.43</td>
</tr>
<tr>
<td>5</td>
<td>k_{23}</td>
<td>32.15</td>
</tr>
<tr>
<td>6</td>
<td>k_{32}</td>
<td>159.85</td>
</tr>
<tr>
<td>7</td>
<td>k_{33}</td>
<td>190.13</td>
</tr>
<tr>
<td>8</td>
<td>k_{34}</td>
<td>188.69</td>
</tr>
<tr>
<td>9</td>
<td>k_{43}</td>
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<tr>
<td>13</td>
<td>k_{55}</td>
<td>83.86</td>
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Table 6.2 Relative identification errors in the identification of horizontal storey stiffness

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<th>Identification Errors (%)</th>
</tr>
</thead>
<tbody>
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<tr>
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<tr>
<td>2</td>
<td>k_{2}</td>
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<tr>
<td>3</td>
<td>k_{3}</td>
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<tr>
<td>4</td>
<td>k_{4}</td>
<td>6.513</td>
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<tr>
<td>5</td>
<td>k_{5}</td>
<td>135.23</td>
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</table>
Table 6.3 Performance indices for local control strategy without Kalman filter

<table>
<thead>
<tr>
<th>Index</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Northridge</th>
<th>Kobe</th>
</tr>
</thead>
<tbody>
<tr>
<td>$J_1$ (peak drift ratio)</td>
<td>0.7049</td>
<td>0.8568</td>
<td>0.8939</td>
<td>0.6767</td>
</tr>
<tr>
<td>$J_2$ (peak acc.)</td>
<td>0.8804</td>
<td>0.9903</td>
<td>0.9438</td>
<td>0.7478</td>
</tr>
<tr>
<td>$J_3$ (rms drift ratio)</td>
<td>0.6797</td>
<td>0.7361</td>
<td>0.6754</td>
<td>0.5850</td>
</tr>
<tr>
<td>$J_4$ (rms acc.)</td>
<td>0.6871</td>
<td>0.7438</td>
<td>0.7068</td>
<td>0.5971</td>
</tr>
<tr>
<td>$J_5$ (control force)</td>
<td>0.0482</td>
<td>0.0566</td>
<td>0.0686</td>
<td>0.0713</td>
</tr>
</tbody>
</table>

Table 6.4 Performance indices for local control strategy with Kalman filter

<table>
<thead>
<tr>
<th>Index</th>
<th>El Centro</th>
<th>Hachinohe</th>
<th>Northridge</th>
<th>Kobe</th>
</tr>
</thead>
<tbody>
<tr>
<td>$J_1$ (peak drift ratio)</td>
<td>0.7224</td>
<td>0.8452</td>
<td>0.8878</td>
<td>0.6676</td>
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<tr>
<td>$J_2$ (peak acc.)</td>
<td>0.8439</td>
<td>0.9187</td>
<td>0.8816</td>
<td>0.7339</td>
</tr>
<tr>
<td>$J_3$ (rms drift ratio)</td>
<td>0.6550</td>
<td>0.6914</td>
<td>0.6324</td>
<td>0.5464</td>
</tr>
<tr>
<td>$J_4$ (rms acc.)</td>
<td>0.6648</td>
<td>0.7114</td>
<td>0.6702</td>
<td>0.5654</td>
</tr>
<tr>
<td>$J_5$ (control force)</td>
<td>0.0476</td>
<td>0.0521</td>
<td>0.0676</td>
<td>0.0674</td>
</tr>
</tbody>
</table>

Table 6.5 Performance indices for global control strategy

<table>
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<tr>
<th>Index</th>
<th>Global control strategy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Index</td>
<td>El Centro</td>
</tr>
<tr>
<td>$J_1$ (peak drift ratio)</td>
<td>0.6274</td>
</tr>
<tr>
<td>$J_2$ (peak acc.)</td>
<td>0.7291</td>
</tr>
<tr>
<td>$J_3$ (rms drift ratio)</td>
<td>0.4549</td>
</tr>
<tr>
<td>$J_4$ (rms acc.)</td>
<td>0.4829</td>
</tr>
<tr>
<td>$J_5$ (control force)</td>
<td>0.0990</td>
</tr>
</tbody>
</table>
Figure 6.1 Vibration control system using semi-active friction dampers: (a) Configuration of control system; (b) Damper mechanical model.

Figure 6.2 Health monitoring system.
Figure 6.3 Integrated vibration control and health monitoring system.
Figure 6.4 Flow chart of system identification process.
Figure 6.5 Damper force flow chart using local control strategy with Kalman filter.
Figure 6.6 Flow chart of vibration control process.
Figure 6.7 Flow chart of damage detection process.

Original building → Long term service and extreme events → Damaged building

Change damper slipping states

Dynamic properties $X_{ii}^0, A_{ii}^0, X_1^0, A_1^0$

Identification equation $(A_i^0)^T k_i^0 = h_i$

Original stiffness parameters $k_0$

Damage detection $\Delta k^p = k^p - k_0$

End

Dynamic properties $X_{ii}^f, A_{ii}^f, X_1^f, A_1^f$

Identification equation $(A_i^f)^T k_i^f = h_i^f$

Damaged stiffness parameters $k^f$

Figure 6.8 Five-storey shear building with semi-active friction dampers.
Figure 6.9 Identification results of mass coefficients without noise contamination.

Figure 6.10 Relative identification errors in natural frequencies.

Figure 6.11 Relative identification errors in modal shapes with known ground motion.
Figure 6.12 Relative identification errors in modal shapes without knowing ground motion.

Figure 6.13 Relative identification errors in stiffness and mass coefficients with 1% noise intensity.

Figure 6.14 Effects of noise level on identification quality with known ground motion
(a) Stiffness coefficients; (b) Mass coefficients.
Figure 6.15 Effects of noise level on identification quality without knowing ground motion (a) Stiffness coefficients; (b) Mass coefficients.

Figure 6.16 Effects of higher modal information on identification quality (a), (b) with four modes of vibration; (c), (d) with five modes of vibration.
Figure 6.17 Relative changes in natural frequencies against stiffness ratio.

Figure 6.18 Average identification errors in stiffness coefficients against stiffness ratio.
Figure 6.19 Time histories of four historical seismic records.

Figure 6.20 Variation of mean vibration reduction factor of displacement, velocity and acceleration response with gain coefficients $G_e$.

Figure 6.21 Comparison of control performance for various control strategies:
(a) Displacement; (b) Velocity; (c) Acceleration.
Figure 6.22 Comparison of actual responses with estimated responses: 
(a) Displacement; (b) Velocity; (c) Acceleration.

Figure 6.23 Comparison of response time histories of top floor: 
(a) Displacement; (b) Velocity; (c) Acceleration.
Figure 6.24 Variation of VRFs with SR using local control strategy with Kalman filter.

Figure 6.25 Damage detection results for single damage event.
Figure 6.26 Damage detection results for double damage event.

Figure 6.27 Damage detection results by sensitivity based approach.
INTINTEGRATED HEALTH MONITORING AND VIBRATION CONTROL OF RETICULATED SHELL

7.1 INTRODUCTION

The concept of the integrated health monitoring and control system using semi-active friction dampers has been presented and applied to a shear building in Chapter 6. The feasibility of the integrated system has also been examined numerically for this simple structure in Chapter 6. The next task is to explore the feasibility of the integrated system to complicated structures such as the reticulated shell in this chapter. Because the configuration of the reticulated shell is much more complicated than that of the shear building, the control algorithm based on the Kalman filter proposed in Chapter 6 for the shear building is not suitable for the vibration control of the reticulated shell. In addition, the identification procedure of the reticulated shell is also different from that of the shear building to some extent. The accuracy of the parameter identification and damage detection for the reticulated shell of complicated configuration is not as good as that of simple shear building which may be improved by using regularization approach such as the L-curve method.

In this chapter, the equation of motion of the reticulated shell without control devices is first established and the dynamic responses are computed to explore the dangerous parts of the shell under seismic excitation. Then a practical approach for incorporating semi-active friction dampers into the reticulated shell is presented.
Based on the dynamic responses of the reticulated shell under earthquake, two damper installation schemes are suggested for seismic mitigation. The equation of motion of the reticulated shell subjected to earthquake and control forces is deduced and a local control strategy only using the local information of semi-active friction dampers is utilized to realize the seismic mitigation. The vibration control performance of two damper installation schemes is compared in order to determine the most appropriate position of dampers. After that, the parameter identification and damage detection using the known additional stiffness provided by semi-active friction dampers are carried out. For parameter identification, the model updating scheme based on adding known stiffness proposed in Chapter 6 is extended for the reticulated shell. The transform matrix of stiffness parameters is deduced based on the structural connectivity and transformation information. In addition, the damage detection method proposed for the shear building in Chapter 6 is also extended for the reticulated shell. The numerical investigation is conducted to examine the performance of integrated health monitoring and vibration control system for parameter identification and damage detection without/with noise contamination. The observations made in this chapter from seismic mitigation, parameter identification and damage detection will be utilized in Chapter 9 for the design of the integrated health monitoring and vibration control system for the reticulated shell.

7.2 SEISMIC RESPONSE CONTROL FOR RETICULATED SHELL

In Chapter 6, the semi-active friction dampers are installed in each floor of the shear building to reduce the dynamic responses under seismic excitation. The semi-active friction dampers also utilized in this chapter to realize the vibration control for the reticulated shell. The seismic responses of the reticulated shell without control are
examined to explore the dangerous parts of the shell under earthquake. Following these observations, the placement of semi-active friction dampers is determined to conduct vibration control.

7.2.1 Seismic responses of reticulated shell

7.2.1.1 Seismic inputs and structural parameters

To evaluate the performance of the reticulated shell under strong earthquake, the three orthogonal components of the El-Centro 1940 earthquake ground acceleration are used. The amplitudes in the three components are adjusted using the same scale with the maximum peak amplitude being $4.0 m/s^2$. The numbers of nodes and elements of the reticulated shell are displayed in Figure 7.1. All the analytical parameters utilized in the dynamic analysis are the same as those adopted in Chapter 5. The equation of motion of the reticulated shell subjected to seismic excitation can be expressed as

$$M\ddot{x} + C\dot{x} + Kx = -ME\ddot{g}$$

(7.1)

$$\ddot{g} = [\ddot{x}_x(t) \ddot{x}_y(t) \ddot{x}_z(t)]^T$$

(7.2)

$$E = [E_x, E_y, E_z]$$

(7.3)

where: $M$, $C$, and $K$ are mass, damping, and stiffness matrices of the reticulated shell respectively; $x$, $\dot{x}$, and $\ddot{x}$ are the displacement, velocity and acceleration vectors of the reticulated shell relative to the ground respectively; $\ddot{g}$ is the ground acceleration and $\ddot{g}_s(t)$ ($s = x, y, z$) is the components of earthquake excitation in three translation directions, $\ddot{x}_x(t), \ddot{x}_y(t)$ and $\ddot{x}_z(t)$ denote the east-west (EW, global x direction), north-south (NS, global y direction) and vertical (VER, global z direction) components of the seismic ground motion respectively; $E$ is the position matrix of
seismic excitation and \( E_s = x, y, z \) represents the position vector of seismic excitation component \( \ddot{x}_s(t) \). The equation of motion of the shell is solved using the Newmark-\( \beta \) method with a time step of 0.002s. The two factors in the Newmark-\( \beta \) method are selected as \( \alpha = 1/2 \) and \( \beta = 1/4 \). The Rayleigh damping assumption is adopted to construct the structural damping matrix, and the damping ratios in the first two modes of vibration of the reticulated shell are set at 0.01. It should be pointed out that the east-west (EW) component of earthquake acts in the global \( x \) direction, north-south (NS) component of earthquake acts in the global \( y \) direction and vertical (Ver) component of earthquake acts in the global \( z \) direction.

7.2.1.2 Seismic responses under earthquake

The maximum axial force and axial stress of members of the reticulated shell subjected to El Centro earthquake are computed and plotted in Figure 7.2. It is seen that the members with large axial forces and stresses under seismic excitation are the radial and circular members within the first three circles. These members have relative larger axial forces and stresses compared to other members of the shell. In addition, Figure 7.3 demonstrates the maximum bending moments of members of the reticulated shell under seismic excitation. Clearly, the bending moment responses of members are quite small compared with the maximum responses of axial force and axial stress. The maximum nodal displacement is also analyzed in this section. For the sake of clarification, only six nodes in the global \( x \) direction shown in Figure 7.1(a) are selected for the display of results. The displacement responses under two dimensional (2D) El Centro earthquakes are also computed and plotted in Figure 7.4 for comparison. The horizontal displacement responses of nodes under 2D and 3D seismic excitation are quite close while the vertical displacement responses of nodes
under 3D seismic excitation are slightly larger than those under 2D excitation. To compare the nodal displacement responses in different directions, one can find that the vertical displacement is much larger than horizontal displacement under multi-dimensional (2D and 3D) seismic excitations. Therefore, the vertical displacement responses are more important for the reticulated shell under seismic excitation and they should be effectively controlled.

By summarizing the dynamic responses of the reticulated shell, one can understand that the responses of bending moment are quite small compared with those of axial force and axial stress. These observations are similar to those made in Chapter 3 from the static analysis as shown in Figure 3.30. Therefore, the static and dynamic responses of the reticulated shell mainly occur in member’s axial direction. The rigid connection of node can be simplified as joint connection. The structural performance of the reticulated shell with rigid connection is quite similar to the reticulated shell with same geometric and physical parameters but simplified as joint connection. The numerical investigation conducted in this chapter also demonstrates that the radial members and circular members have relative large axial deformation than the skew members. Because the semi-active friction dampers reduce the dynamic responses by dissipating vibrant energy through the slipping process, satisfactory control performance can be achieved if dampers are placed where they have large slippage during earthquake. Therefore, the semi-active friction dampers may be incorporated into some of radial and circular members to achieve satisfactory seismic mitigation effects.
7.2.2 Equation of motion of controlled reticulated shell

The semi-active friction dampers can be incorporated into the reticulated shell to abate the structural dynamic responses. Considering the configuration of the reticulated shell, a semi-active friction damper with an axial brace can be connected to a structural member in parallel as shown in Figure 7.5. The semi-active friction damper connects the shell node through joint connection in axial direction of the concerned member, which means that the control forces provided by damper directly act on joint connection in the member’s axial direction. If \( k_d \) is the brace stiffness of semi-active friction damper, the control force of semi-active friction damper connecting two nodes \((j \text{ and } k)\) of the member can be expressed as

\[
u_d = k_d(u_k - u_j - e) = S \cdot \text{Sgn}(\dot{e})\quad (7.4)\]

where \( u_j \) and \( u_k \) are the displacement of two ends of the damper; \( e \) denotes the damper slippage; \( S \) is the slipping force.

In the local coordinate system (LCS), the control force of the \( i \)th semi-active friction damper \( f_e^{(i)} \) can be deduced following structural theory

\[
f_e^{(i)} = \mathbf{K}_e^{(i)} \delta_e^{(i)} + \mathbf{b}_e^{(i)} \dot{e}^{(i)} \quad (7.5)
\]

where

\[
f_e^{(i)} = [f_{j,u} \ 0 \ 0 \ f_{k,u} \ 0 \ 0]^T \quad (7.6)
\]

\[
\mathbf{K}_e^{(i)} = k_e^{(i)}
\]

\[
\delta_e^{(i)} = [u_j \ v_j \ w_j \ u_k \ v_k \ w_k]^T \quad (7.8)
\]
\[ b_e^{(i)} = k_d^{(i)}[1 \ 0 \ 0 \ -1 \ 0]^T \]  \hspace{1cm} (7.9)

In Equation (7.6) \( f_{js}^{(k)} (s = u) \) is the force components of node \( j \) in the axial direction;

\( K_e^{(i)} \) is the stiffness matrix of the \( i \)th semi-active friction damper in local coordinate system; \( k_d^{(i)} \) is the brace stiffness coefficient of the \( i \)th semi-active friction damper;

\( \delta_e^{(i)} \) is the node displacement vector of the \( i \)th semi-active friction damper in LCS and \( s_j^{(k)} (s = u, v, w) \) is the displacement components of node \( j \) in three translation directions; \( b_e^{(i)} \) is the coefficient vector of slipping displacement of the \( i \)th semi-active friction damper in LCS.

Following the principles of finite element method and constructing the coordinate transformation matrix \( T_e^{(i)} \), the force of the \( i \)th semi-active friction damper \( \bar{f}_e^{(i)} \) in global coordinate system (GCS) is

\[ \bar{f}_e^{(i)} = K_e^{(i)}\bar{\delta}_e^{(i)} + \bar{b}_e^{(i)}e^{(i)} \]  \hspace{1cm} (7.10)

where

\[ \bar{f}_e^{(i)} = T_e^{(i)T}f_e^{(i)} \]  \hspace{1cm} (7.11)

\[ \bar{K}_e^{(i)} = T_e^{(i)T}K_e^{(i)}T_e^{(i)} \]  \hspace{1cm} (7.12)

\[ \bar{\delta}_e^{(i)} = T_e^{(i)T}\delta_e^{(i)} \]  \hspace{1cm} (7.13)

\[ \bar{b}_e^{(i)} = T_e^{(i)T}b_e^{(i)} \]  \hspace{1cm} (7.14)

\[ T_e^{(i)} = \begin{bmatrix} \lambda & 0 \\ 0 & \lambda \end{bmatrix} \]  \hspace{1cm} (7.15)

in which \( \lambda \) is the rotation matrix as expressed in Equation (3.19) whose elements represent the direction cosines of local axes in GCS; \( \bar{K}_e^{(i)} \) is the stiffness matrix of the \( i \)th semi-active friction damper in GCS; \( \bar{\delta}_e^{(i)} \) is the node displacement vector of the
ith semi-active friction damper in GCS; $\mathbf{b}_{i}^{(i)}$ is the coefficient vector of slipping displacement of the ith semi-active friction damper in GCS.

Premultiplying the position matrix of element freedom $\mathbf{T}_{e}^{(i)}$ to Equation (7.10) yields

$$
\mathbf{T}_{e}^{(i)} \bar{f}_{e}^{(i)} = \mathbf{T}_{e}^{(i)T} \mathbf{K}_{e}^{(i)} \mathbf{T}_{e}^{(i)} \mathbf{T}_{e}^{(i)T} \mathbf{\delta}_{e}^{(i)} + \mathbf{T}_{e}^{(i)T} \mathbf{b}_{e}^{(i)} \mathbf{e}^{(i)}
$$

(7.16)

The control force $\bar{f}_{i}^{(i)}$ of the ith semi-active friction damper to the reticulated shell in GCS is

$$
\bar{f}_{i}^{(i)} = \mathbf{K}_{i}^{(i)} \mathbf{\delta}_{i}^{(i)} + \mathbf{b}_{i}^{(i)} \mathbf{e}^{(i)}
$$

(7.17)

where

$$
\bar{f}_{i}^{(i)} = \mathbf{T}_{e}^{(i)T} \bar{f}_{e}^{(i)}
$$

(7.18)

$$
\mathbf{K}_{i}^{(i)} = \mathbf{T}_{e}^{(i)T} \mathbf{K}_{e}^{(i)} \mathbf{T}_{e}^{(i)}
$$

(7.19)

$$
\mathbf{\delta}_{i}^{(i)} = \mathbf{T}_{e}^{(i)T} \mathbf{\delta}_{e}^{(i)}
$$

(7.20)

$$
\mathbf{b}_{i}^{(i)} = \mathbf{T}_{e}^{(i)T} \mathbf{b}_{e}^{(i)}
$$

(7.21)

in which the position matrix of element freedom $\mathbf{T}_{e}^{(i)}$ represents the relation between element DOFs and structural DOFs; $\mathbf{K}_{i}^{(i)}$ is the contribution of the stiffness matrix of the ith semi-active friction damper on the structural global stiffness matrix; $\mathbf{\delta}_{i}^{(i)}$ is the component vector of the structural displacement from the ith semi-active friction damper; $\mathbf{b}_{i}^{(i)}$ is the coefficient matrix of slipping displacement of the ith semi-active friction damper in GCS.

To collect the contribution of all the semi-active friction dampers installed in the reticulated shell, the whole control force in the GCS $\mathbf{f}_{c}$ can be written as

$$
\mathbf{f}_{c} = \sum_{i=1}^{n_d} \bar{f}_{i}^{(i)} = \sum_{i=1}^{n_d} \left( \mathbf{K}_{i}^{(i)} \mathbf{\delta}_{i}^{(i)} + \mathbf{b}_{i}^{(i)} \mathbf{e}^{(i)} \right)
$$

(7.22)
where \( nd \) is the number of semi-active friction dampers. By taking Equations (7.11) and (7.18) into consideration, the control force can be expressed as

\[
f_c = \sum_{i=1}^{nd} \mathbf{f}_i^{(i)} = \sum_{i=1}^{nd} \mathbf{T}_c^{T(i)} \mathbf{T}_u^{i} \mathbf{f}_c = \mathbf{H} \mathbf{u} \tag{7.23}
\]

\[
\mathbf{H}^c = [\mathbf{T}_c^{T(1)} \mathbf{T}_u^{(1)}, \mathbf{T}_c^{T(2)} \mathbf{T}_u^{(2)}, ..., \mathbf{T}_c^{T(nd)} \mathbf{T}_u^{(nd)}] \tag{7.24}
\]

\[
\mathbf{u} = [\mathbf{f}_c^{(1)}, \mathbf{f}_c^{(2)}, ..., \mathbf{f}_c^{(nd)}]^T \tag{7.25}
\]

where \( \mathbf{u} \) is the control force of semi-active friction dampers; \( \mathbf{H}^c \) is the influence matrix reflecting the location of the control forces. The equation of motion of the reticulated shell with semi-active friction dampers subjected to seismic excitation can be expressed as

\[
\mathbf{M} \ddot{\mathbf{x}} + \mathbf{C} \dot{\mathbf{x}} + \mathbf{K} \mathbf{x} = -\mathbf{M} \mathbf{E} \ddot{\mathbf{x}}_g + \mathbf{H} \mathbf{u} \tag{7.26}
\]

\[
\ddot{\mathbf{x}}_g = [\ddot{\mathbf{x}}_x(t), \ddot{\mathbf{x}}_y(t), \ddot{\mathbf{x}}_z(t)]^T \tag{7.27}
\]

\[
\mathbf{E} = [\mathbf{E}_x, \mathbf{E}_y, \mathbf{E}_z] \tag{7.28}
\]

The meanings of symbols in Equation (7.26) are the same as those in Equation (7.1).

### 7.2.3 Control strategy and damper installation scheme

In the seismic mitigation of the shear building discussed in Chapter 6, the local control strategy with the Kalman filter is developed for semi-active friction dampers. Five accelerometers are installed on the building, each for one floor, to measure the acceleration responses of building subjected to earthquake ground excitation in the case of vibration control. The displacement and velocity responses can be effectively predicted based on the Kalman filter using the measured acceleration responses. However, this procedure cannot be successfully applied to the vibration control of the reticulated shell. This is because the response prediction using the Kalman filter
needs to solve the Riccati equation (See Equation (6.59)). The numerical investigation carried out in this study demonstrates that the solution precision of the Riccati equation for the reticulated shell with a great number of DOFs is not good enough. In addition, in real vibration control and health monitoring systems, it is impossible and impractical to install a great number of accelerometers. In this regard, control strategy based on local feedback information obtained from limited sensors is adopted to realize the seismic mitigation of the reticulated shell. The local control strategy proposed by Chen and Chen (2002) which is adopted in the seismic mitigation of the shear building in Chapter 6 is applied to the vibration control of the reticulated shell. Because of the large number of friction dampers adopted in seismic mitigation, it is impossible and inconvenient to set different gain coefficients for different dampers. For simplicity, the gain coefficients of all the dampers are taken to be the same value. The results from numerical investigation demonstrate that no satisfactory control effects can be achieved. This is because the control strategy proposed by Chen and Chen (2002) is mainly based on the slipping states of semi-active friction dampers while the slipping states of dampers at different positions demonstrate obvious discrepancy. Therefore, the same gain coefficient applied to all the semi-active friction dampers cannot achieve satisfactory control efficacy. To avoid this shortcoming, the local feedback control strategy for semi-active friction damper proposed by Akaby and Aktan (1990) is adopted for the seismic mitigation of the reticulated shell. This control strategy aims to change the slipping force of friction damper for more energy dissipating and better control performance

\[ u_t = \begin{cases} 
S + \Delta S & \text{slipping stage} \\
S - \Delta S & \text{stick stage} 
\end{cases} \quad (7.29) \]

\[ \Delta S = \alpha \cdot \mu N_{\text{max}}^d \quad (7.30) \]
where $S$ is the slipping force of semi-active friction damper which can be set at zero at the beginning; $\Delta S$ is the slipping force increment; $\mu$ is the friction coefficient of semi-active friction damper; $N_{\text{max}}^d$ is the maximum clamping force which can be provided by the semi-active friction damper (See Equation (6.64)); $\alpha$ is the increment ratio of slipping force. The control strategy in Equation (7.29) reveals that if the damper is in the loading process ($d \cdot \dot{d} > 0$), the member’s axial force gradually increases and the increment of slipping force can frustrate this increase and reduce member responses. Inversely, if the damper is in the unloading process ($d \cdot \dot{d} \leq 0$), the axial force of the member gradually decreases and the reduction of slipping force can accelerate this reducing process. In the real application, a displacement transducer and a force transducer are required to be incorporated with each semi-active friction damper to collect the local dynamic information for control process. As mentioned above, the semi-active friction damper connects the shell node through joint connection in the member’s axial direction. Therefore, the control forces provided by friction damper directly act on joint connection in the member’s axial direction.

As mentioned before, satisfactory control performance can be achieved if dampers are placed where they have large slippage during earthquake. Some of radial and circular members can be selected for the incorporation of semi-active friction dampers for seismic mitigation because these members have relative large axial deformation than the skew members. Practically, more dampers can achieve better control performance. While for the complicated reticulated shell with a great number of members, it is uneconomical to install too many dampers for response control. In addition, the analysis on seismic responses carried out in the former section reveals
that some regions of the shell have strong dynamic responses while other parts do not have strong member forces and nodal displacement. Therefore, only limited number of semi-active friction dampers is utilized to reduce the dynamic responses of some dangerous parts. In this regard, it is decided to install 48 semi-active friction dampers in the reticulated shell and two damper installation schemes are designed to compare their control performance as shown in Figure 7.6. The numerical investigation demonstrates that the axial deformation of the radial members is slightly larger than that of circular members under both static and seismic loads. Therefore, the damper incorporated into the radial member may have relative larger slippage to obtain better control performance. Based on this observation, the damper installation scheme No.1 is presented in which 48 dampers are incorporated into the radial members as shown in Figure 7.6 (a). As mentioned above, the members within the first three circles have relative larger dynamic responses than other parts under earthquake. The seismic responses of these members should be effectively controlled. Therefore, the damper installation scheme No.2 is presented in which 48 friction dampers are incorporated into radial and circular members within the first three circles as shown in Figure 7.6 (b). One of important properties of semi-active friction damper is its brace stiffness which may have important influence on the control performance. The stiffness ratio \(SR\) of semi-active friction damper for the reticulated shell is defined as

\[
SR = \frac{K_d}{K_s}
\]

(7.31)

where \(K_d\) is the additional stiffness from the brace when semi-active friction damper is in sticking state; \(K_s\) is a prescribed stiffness value which is adopted as axial stiffness of radial member in this study.
7.2.4 Control performance

The seismic mitigation of the reticulated shell is carried out in this section using the local feedback control strategies expressed in Equation (7.29). The seismic excitation used is the El-Centro 1940 earthquake ground acceleration with 3D components, and the amplitudes of the three components are adjusted using the same scale with the maximum peak amplitude being $4.0 \text{m/s}^2$. All the mass parameters of the reticulated shell are the same as those adopted in Chapter 5 from the theoretical model. The stiffness matrix is constructed using the stiffness parameters (EA) identified by using the proposed system identification algorithm at 0.1% noise level. The identification process will be illustrated in the section of system identification in detail. The equation of motion of the shell is solved using the Newmark-$\beta$ method with a time step of 0.002s. The two factors in the Newmark-$\beta$ method are selected as $\alpha = 1/2$ and $\beta = 1/4$. The Rayleigh damping assumption is adopted to construct the structural damping matrix, and the damping ratios in the first two modes of vibration of the reticulated shell are set at 0.01. The control performance is evaluated in terms of a vibration reduction factor ($VRF$) defined in Equation (6.79). Only the member internal forces and nodal displacement, which affect the safety of the reticulated shell during earthquake excitation, are considered here when examining the control performance. The acceleration response of the shell is not considered in assessing the control performance. The $SR$s of all the semi-active friction dampers are determined as 1.2 for both damper schemes through numerical study. The determination of optimal $SR$ value will be discussed later. The results from parameter investigation suggest that the optimal increment ratios expressed in Equation (7.30) are taken as $\alpha = 0.00035$ and $\alpha = 0.0004$ for damper schemes No.1 and No.2 respectively.
As discussed before, the regions within the first three circles are the most dangerous parts of the reticulated shell under El Centro earthquake because the members and nodes within these regions have larger member forces and displacement than other parts. Therefore, the seismic responses of these regions are examined in detail after installing semi-active friction dampers. Figure 7.7 demonstrates the displacement $VRF$s of nodes within the first three circles of the shell under El Centro earthquake. It is seen that the maximum nodal displacement is effectively reduced at most of the places within the first three circles based on damper scheme No.2. The overall performance of the semi-active friction dampers adopting scheme No.2 on reducing displacement responses in $x$ and $z$ directions is satisfactory while the control effects on displacement responses in $y$ direction are slightly worse. Compared with damper scheme No.2, the control performance of damper scheme No.1 is worse to some extent especially for vertical displacement. Because the displacement responses in $z$ direction are significantly larger than those in horizontal directions, the control performance in vertical direction is most important for improving the structural safety under earthquake. Therefore, it is concluded that the control performance of damper scheme No.2 is better than that of damper scheme No.1 for displacement responses. The $VRF$s of axial forces of members within the first three circles are listed in Table 7.1 for both two damper schemes. For damper scheme No.1, only the axial forces of radial members are effectively suppressed while the response mitigation efficacy of circular members and skew members is not satisfactory. The data for damper scheme No.2 demonstrate that the axial forces of radial, circular and skew members within the first three circles are substantially suppressed compared to those of scheme No.1. The comparison between these two damper schemes clearly
indicates that the control performance of scheme No.2 is much better than that of scheme No.1.

Demonstrated in Figures 7.8 (a)-(c) are the displacement response time histories of shell vertex (node 1) adopting damper scheme No.2. The actual displacement time histories of shell vertex in various directions without control are also illustrated in Figures 7.8 (a)-(c). The comparison between the two sets of time histories clearly reveals that the installation of semi-active friction dampers can suppress the displacement responses under seismic excitation. The comparison of displacement mitigation in various directions reveals the best displacement reduction in the vertical direction while the control effects on horizontal displacement responses are worse to some extent. Considering the deformation of the reticulated shell under earthquake is mainly contributed by the vertical displacement, the control effects of nodal displacement using semi-active friction dampers are satisfactory.

As mentioned in Chapter 6, the feasibility of the proposed integrated control and monitoring procedure depends on the brace (damper) stiffness for both vibration control and model updating. Therefore, the optimal value for the brace stiffness should be determined to pursue better control and identification effects. Figure 7.9 displays the variation of $VRF_s$ of nodal displacement and axial forces of members with $SR$ based on damper scheme No.2. It can be seen that the vibration reduction factors for nodal displacement and member’s axial force increase when the stiffness ratio increases from 0.0 to about 0.8. Afterwards, the vibration reduction factors increase only slightly with increasing stiffness ratio. When it reaches above 1.2, the $SR$ has almost no effect on the vibration reduction factors. The similar results are
observed from responses of other nodes or members. As a result, the optimum $SR$ for seismic response control of the reticulated shell is selected as 1.2.

7.3 PARAMETER IDENTIFICATION OF RETICULATED SHELL

The parameter identification by using semi-active friction dampers is developed and applied to a shear building in Chapter 6 for the construction of the integrated health monitoring and vibration control system. This integrated system is also applied in this chapter for vibration control, parameter identification and damage detection. The vibration control using semi-active friction dampers are carried out in the former section. This section will investigate the parameter identification of the reticulated shell using semi-active friction dampers. As discussed in Chapter 3, the bending responses (bending moments and bending stresses) of shell members under dead loads are quite small compared to axial responses (axial forces and axial stresses). Similar observations can be made from the seismic analysis of the reticulated shell carried out in this chapter. By summarizing the static and dynamic responses of the reticulated shell, one can understand that the static and dynamic responses of the reticulated shell mainly occur in member’s axial direction and the bending responses are negligible in comparison with the axial responses. The rigid connection of node can be simplified as joint connection. The axial stiffness of the shell member is quite important in determining the shell performance under external excitations. The structural performance of the reticulated shell with rigid connection is quite similar to the reticulated shell with the same geometric dimension and physical parameters but simplified as joint connection. Therefore, in the parameter identification of the reticulated shell carried out in this chapter, the original rigid connection of the node is simplified as joint connection to simplify the analytical model. Only the axial
stiffness parameters of the shell members are identified using the proposed algorithm in this study. The axial stiffness parameter is selected as the product of Young’s modulus and cross section area (EA) of a member. As discussed before, the damper installation scheme No.2 presents better control performance than the scheme No.1. Therefore, the damper scheme No.2 of semi-active friction dampers is adopted to provide the known additional stiffness in the parameter identification and damage detection. In this section, transform matrix of stiffness parameters is first deduced based on the finite element model of the reticulated shell. Then the parameter identification without/with noise contamination is carried out to estimate the stiffness parameters (EA).

7.3.1 Identification equation of stiffness parameters

Based on the algorithm proposed in Chapter 6, the identification equation of the elements of global stiffness matrix $K$ of the reticulated shell can be expressed as (See Equation (6.12))

$$A_k \cdot k_0 = h_k$$  \hspace{1cm} (7.32)

where

$$A_k = X_{0k}^T \otimes X_k^T$$  \hspace{1cm} (7.33)

$$k_0 = [k_{i1} \cdots k_{iN} | k_{j1} \cdots k_{j2N} | \cdots | k_{N1} \cdots k_{NN}]^T$$  \hspace{1cm} (7.34)

$$h_k = [h_{i1}^k \cdots h_{iN}^k | h_{j1}^k \cdots h_{j2N}^k | \cdots | h_{N1}^k \cdots h_{NN}^k]^T$$  \hspace{1cm} (7.35)

in which $A_k$ is the coefficient matrix of size $N_e^2 \times N^2$ defined in Equation (7.33) in which $\otimes$ is the Kronecker product; the vector $k_0$ is of length $N^2$ and its element $k_{ij}$ corresponds to the $(i,j)$th element of $K$; and the vector $h_k$ is of length $N_e^2$ (See Equation 6.11); $N$ is the degrees of freedom of the reticulated shell; $N_e$ is the
available number of measured natural frequencies and model shapes of the reticulated shell in identification process; $X_{0k}, X_i$ are the measured $N \times N_e$ modal matrix of the shell without and with additional stiffness respectively.

For the reticulated shell, the element stiffness matrix of the $m$th member in the LCS $K_{c}^{(m)}$ can be expressed as

$$K_{c}^{(m)} = S_{{k,c}}^{(m)}$$  (7.36)

$$S_{{k,c}}^{(m)} = \frac{\partial K_{c}^{(m)}}{\partial \gamma^{(m)}}$$  (7.37)

where $\gamma^{(m)}$ is the stiffness parameter of the $m$th element which is selected as EA in this study; $S_{{k,c}}^{(m)}$ is the sensitivity matrix of the $m$th element to the stiffness parameter $\gamma^{(m)}$. The element stiffness matrix $\bar{K}_{c}^{(m)}$ in GCS can be expressed using the coordinate transformation matrix of the $m$th element $T_{a}^{(m)}$

$$\bar{K}_{c}^{(m)} = T_{c}^{(m)T} S_{{k,c}}^{(m)} T_{a}^{(m)} \gamma^{(m)}$$  (7.38)

The contribution of the stiffness of the $m$th element $\bar{K}^{(m)}$ to the global stiffness matrix $K$ is

$$\bar{K}^{(m)} = T_{c}^{(m)T} S_{{k,c}}^{(m)} T_{a}^{(m)} \gamma^{(m)} = S_{{k,c}}^{(m)} \gamma^{(m)}$$  (7.39)

$$S_{{k,c}}^{(m)} = T_{c}^{(m)T} S_{{k,c}}^{(m)} T_{a}^{(m)} T_{c}^{(m)} = T_{c}^{(m)T} S_{{k,c}}^{(m)} T_{c}^{(m)}$$  (7.40)

$$T_{c}^{(m)} = T_{a}^{(m)} T_{c}^{(m)}$$  (7.41)

where $S_{{k,c}}^{(m)}$ is the sensitivity matrix of the global stiffness matrix $K$ to the stiffness parameter $\gamma^{(m)}$ of the $m$th element; $T_{c}^{(m)}$ is the products of coordinate transformation matrix $T_{a}^{(m)}$ and position matrix $T_{c}^{(m)}$ of the $m$th element (See Equation (7.41)).
To express the matrices $\mathbf{K}^{(m)}$ and $\mathbf{S}_{K,\gamma}^{(m)}$ in the column vector format through row spread ($rs(\cdot)$), Equation (7.39) can be rewritten by

$$[rs(\mathbf{K}^{(m)})]^{T} = [rs(\mathbf{S}_{K,\gamma}^{(m)})]^{T} \cdot \gamma^{(m)}$$

(7.42)

Furthermore, the global stiffness matrix can be expressed as

$$\mathbf{K} = \sum_{m=1}^{ne} \mathbf{K}^{(m)} = \sum_{m=1}^{ne} \mathbf{S}_{K,\gamma}^{(m)} \cdot \gamma^{(m)}$$

(7.43)

where $ne$ is the number of total elements of the reticulated shell.

The elements of the global stiffness matrix can be expressed in the column vector form $\mathbf{k}_{0}$ (See Equation (7.34))

$$\mathbf{k}_{0} = [rs(\mathbf{K})]^{T}$$

(7.44)

Taking Equations (7.43) and (7.42) into consideration yields

$$\mathbf{k}_{0} = \sum_{m=1}^{ne} [rs(\mathbf{K}^{(m)})]^{T} = \sum_{m=1}^{ne} ([rs(\mathbf{S}_{K,\gamma}^{(m)})]^{T} \cdot \gamma^{(m)})$$

(7.45)

Rewriting Equation (7.45) in the matrix form yields

$$\mathbf{k}_{0} = \mathbf{T}_{k} \mathbf{\gamma}_{0}$$

(7.46)

where

$$\mathbf{T}_{k} = [rs_{K,\gamma}^{(1)}, rs_{K,\gamma}^{(2)}, \ldots rs_{K,\gamma}^{(m)}, \ldots rs_{K,\gamma}^{(mn)}]$$

(7.47)

$$\mathbf{\gamma}_{0} = [\gamma^{(1)}, \gamma^{(2)}, \ldots \gamma^{(m)}, \ldots \gamma^{(mn)}]^{T}$$

(7.48)

If the stiffness parameters of all the members are required to be identified, the number of unknown quantities in the detection process for stiffness parameters and elements of global stiffness matrix $\mathbf{K}$ are $ne$ and $N \times N$ respectively. Following the relationship between global stiffness matrix $\mathbf{K}$ and stiffness parameters (EA) of each member, the transform matrix of stiffness parameter $\mathbf{T}_{k}$ for the original shell has the
The identification equation of stiffness parameter of the reticulated shell is
\[(A_i T_k)k_0^p = h_k\]  (7.49)
where $A_i T_k$ is the coefficient matrix of stiffness identification equation with the size of $N_e \times ne$.

7.3.2 Parameter identification without noise contamination

Since noise contamination is not considered, the natural frequencies and mode shapes of the reticulated shell are obtained through eigenvalue analysis as the exact values for the parameter identification. The natural frequency matrix $\Lambda_{nk}$ and the mode shape matrix $X_{nk}$ of the reticulated shell without additional stiffness (the clamping forces of all the semi-active friction dampers are set at zero) are computed. The $SR$ of all the semi-active friction dampers for parameter identification is selected as the optimal value determined through vibration control ($SR=1.2$). The natural frequency matrix $\Lambda_i$ and the mode shape matrix $X_i$ of the shell with the additional stiffness (all the dampers are in sticking state) are also computed. Let us now identify the stiffness parameters (EA) directly using Equation (7.49), the number of mode shapes of the shell, $eN_e$, used for the identification of stiffness parameters should meet the following condition if one wants to avoid the underdetermined identification problem
\[N_e \geq \sqrt{ne}\]  (7.50)
For the reticulated shell, stiffness parameters of 408 structural members (all the member exclude 48 circular members on the sixth circle) are required to be identified. The number of modes of vibration should be 21 if one wants to avoid the
underdetermined identification problem. The stiffness parameters of the reticulated shell without introducing noise contamination are identified adopting the proposed algorithm using the first 21 natural frequencies and mode shapes. The identification error in the $i$th stiffness parameter is defined as the absolute value of the difference between the identified $i$th stiffness parameter and the actual $i$th stiffness parameter and then divided by the actual $i$th stiffness parameter. In addition, the identification accuracy in the $i$th stiffness parameter is defined as the difference between 100 and the identification error in the $i$th stiffness parameter (in percent). The identification accuracy of stiffness parameters is plotted in Figure 7.10 which demonstrates that the stiffness parameters can be effectively determined without any error.

7.3.3 Effects of noise contamination

As mentioned in Chapter 6, the natural frequencies and mode shapes are often obtained with noise contamination. Therefore, the effects of noise on the identification quality of stiffness parameters using the proposed model updating scheme are examined in this section. In this regard, the modal information of the reticulated shell is identified based on the known excitation. A vibration exciter is assumed to be installed on the node 23 at the first circle (the position of node 23 can be seen in Figure 7.1(a)) to provide the external excitation force. The external excitation force is assumed to be a white noise random process with a peak value of 5.4KN acting on the node 23 in the vertical direction. The sampling frequency of 500Hz and the time duration of 200 seconds are used in the dynamic analysis. The acceleration responses of all the nodes under this known input excitation are then computed. The measurement noise, which is also assumed to be a white noise random process, is simulated and added to the acceleration response according to a
given noise intensity. The contaminated acceleration responses are finally used to identify the natural frequencies and mode shapes and to assess the effects on the identification quality of stiffness parameters. The noise intensity is defined as the ratio of the root mean square (RMS) of the noise to the RMS of the acceleration response

\[
\text{Noise intensity} = \frac{\text{RMS}(\text{noise})}{\text{RMS}(\text{acceleration response})} \times 100\%
\]

(7.51)

For the technique using the known excitation, the frequency response functions (FRF) of all the shell nodes are first computed to determine impulse response functions. The Eigensystem Realization Algorithm is then applied to the impulse response functions to obtain the modal information (Juang 1994). The number of adopted natural frequencies and mode shapes in identification process is 21 and the SR of all the semi-active friction dampers is assumed 1.2. Three noise intensities, 0.1%, 0.5% and 1%, are introduced to the acceleration responses of the shell to assess the effects of noise contamination on the identification quality of the stiffness parameters.

To improve the identification accuracy of stiffness parameters, the regularization technique is introduced in the identification process. The identification Equation (7.49) can be written in a general form

\[
A\theta = b
\]

(7.52)

where \(A\) is the coefficient matrix \(A_kT_k\) of identification equation of stiffness parameter; \(\theta\) is the stiffness parameter \(k_0^p\); \(b\) is the coefficient vector \(h_k\) in Equation (7.49). The basic idea of regularization is to define the regularized solution \(\theta(\lambda)\) as the minimizer of the weighted combination of the residual norm and the side constraint (See Equation (2.19))
\[ \|A\theta - b\|_2^2 + \lambda \|L\theta - d\|_2^2 \]  

(7.53)

In this chapter, the L-curve method is applied to solve identification equation by using the toolbox developed by Hansen (1994). The L-curve method defines an upper bound for the side constraint and minimizes the residue expressed in Equation (7.54) to obtain the optimal regularization parameter \( \lambda_{\text{opt}} \) and determine regularized solution \( \theta(\lambda_{\text{opt}}) \) (See Equation (2.37))

\[ \min_\theta \|A\theta - b\|_2 \text{ subjected to: } \|L\theta - d\|_2 \leq \gamma \]  

(7.54)

The identification errors of stiffness parameters at 0.1% noise level is computed using the L-curve method with an optimal regularization parameter \( \lambda_{\text{opt}} = 9.72 \times 10^9 \) as listed in Table 7.2. The results demonstrate that most of the stiffness parameters can be identified under 0.1% noise contamination. It is seen that about 65% of the structural members have identification errors no more than 10%. The members have identification errors no more than 20% is about 87%. The identification errors of stiffness parameters at 0.5% noise level is also computed using the L-curve method with an optimal regularization parameter \( \lambda_{\text{opt}} = 9.62 \times 10^9 \) and the corresponding results are listed in Table 7.3. It is seen that about 46% of the structural members have identification errors no more than 10%. In addition, the members have identification errors no more than 20% is about 76%. To compare the identification errors under 0.1% and 0.5% noise intensities, one can find that with the increase of noise intensity, the detection accuracy of stiffness parameters rapidly reduced. The identification errors under 1% noise contamination is also analyzed and listed in Table 7.4. It can be seen that about 12% of the structural members have identification errors more than 50%. These observations
reveal that the identification errors of stiffness parameters of the reticulated shell under 0.5% and 1% noise contamination are unsatisfactory.

It should be pointed out that the above parameter identification is carried out with the $SR$ value of 1.2. Further numerical investigation is conducted to determine the optimal value of $SR$ for parameter identification of the reticulated shell under noise contamination. The results demonstrate that the optimal values of $SR$ under 0.1% noise intensity are about 1.0. The further increase of $SR$ cannot effectively improve the identification accuracy. As mentioned before, the investigation on control performance conducted in the former section suggests an optimal $SR$ value of 1.2 for satisfactory control performance. This means a compromise should be taken into account to obtain the rational $SR$ value of semi-active friction dampers for both vibration control and system identification of the reticulated shell. In this chapter, the optimal $SR$ value is adopted as 1.2 for the integrated system.

### 7.4 Damage Detection of Reticulated Shell

By adopting the same procedure as used for the system identification and considering the connectivity information of structural members, the stiffness identification equation for the damaged reticulated shell can be derived as (also see Equation (6.67))

$$A^d_k \cdot k^d = h^d_k \tag{7.55}$$

where

$$k^d = [k^d_{i1}, k^d_{i2}, \ldots, k^d_{ij}, \ldots, k^d_{NN}]^T \quad (abs(i-j) \leq 1) \tag{7.56}$$

$$h^d_k = [h^d_{11}, \ldots, h^d_{i1}, h^d_{12}, \ldots, h^d_{i2}, \ldots, h^d_{NN,1}, \ldots, h^d_{NN,N}]^T \tag{7.57}$$
where $A_i^d$ is obtained from $X_{0k}^d \otimes X_i^d$ (See Equation 6.69); the vector $k^d$ is of length $N^2$ and its element $k_{ij}^d$ corresponds to the $(i,j)$th element of global stiffness matrix of damaged shell; the vector $h_i^d$ is of length $N_e^2$.

For the reticulated shell considered in this study, the relationship between the stiffness parameters (EA) of members and the elements of the stiffness matrix is established to obtain the transformation matrix $T_i$ (See Equation (7.47)). The stiffness parameters of the damaged shell, $k^p$, can then be obtained by finding the following minimum norm least-square solution

$$ k^p = (A_i^d T_i)^{'} h_i^d $$

(7.58)

Once the stiffness parameters of the damaged shell are found, the damage location and severity of the damaged shell can be determined by

$$ \Delta k^p = k^p - k_0^p $$

(7.59)

where $k_0^p$ is the stiffness parameters vector of the original shell obtained from the system identification.

The feasibility of the damage detection scheme in the proposed integrated procedure for identifying different damage events of the reticulated shell is examined in this section. The effects of noise contamination on the damage detection quality are also assessed. The stiffness parameter is selected as the product of Young’s modulus and cross section area of member (EA). Two damage scenarios are taken into consideration in this section: (1) 20% axial stiffness loss at the member 1; (2) 20%, 30% and 40% axial stiffness loss at member 1, 7 and 49 respectively. The locations of damaged members are plotted in Figure 7.1 (b). Three noise intensities, 0.1%, 0.5%, and 1%, are considered in the damage detection compared with the case
without noise contamination. In the case of damage detection, the clamping forces of the semi-active friction dampers are set at zero level first to create the status of the damaged shell without any additional stiffness. The clamping forces of the semi-active friction dampers are then set to the maximum value to ensure all the semi-active friction dampers are in sticking states when the damaged shell is subjected to external excitation so as to create the status of the damaged shell with additional known stiffness. The known external excitation used is the same as that used in the model updating. The acceleration responses of all the nodes to the input excitation are computed for the damaged shell with and without additional stiffness. The measurement noise is simulated and added to the acceleration response according to a given noise intensity. The contaminated acceleration responses are then used to identify the natural frequencies and mode shapes, to detect the damage severity and location, and to assess the effects on the quality of damage detection. The first 21 natural frequencies and mode shapes are utilized in the detection process. The $SR$ value of all the semi-active friction dampers are adopted 1.2. The L-curve method is also utilized in the detection process.

The damage size, which is defined as the absolute value of the difference between the detected stiffness parameter and the original stiffness parameter divided by the original stiffness parameter, is computed for the damage events, and the results are displayed in Figures 7.11 and 7.12 for damage scenarios 1 and 2. It can be seen that without measurement noise, the proposed detection scheme can accurately determine the damage location and size. The damage scenario 2 with three damaged members is adopted to examine the effects of noise contamination on detection quality. Displayed in Figure 7.13 are the detection results of the first 50 members for the
damage scenario 2 without/with noise contamination. As shown in Figure 7.13, the damage location can be roughly determined by comparing the detected damage sizes of all the members. With the introduction of measurement noise, the damage size detected is less accurate. The further numerical investigation reveals that under 0.5% and 1% noise intensities, the detection errors of some members are quite large and the damage location cannot be accurately determined.

7.5 SUMMARY

The integrated procedure for health monitoring and vibration control using semi-active friction dampers proposed in Chapter 6 for the shear building is applied to the reticulated shell in this chapter. Owing to the difference in structural arrangement between the shear building and the reticulated shell, the process of vibration control, parameter identification and damage detection using integrated system is also different to some extent. The vibration control of the reticulated shell is first carried out based on the presented damper installation schemes. After that, the parameter identification and damage detection using the known additional stiffness provided by semi-active friction dampers are conducted.

The seismic responses of the reticulated shell without control are examined to explore the dangerous parts of the shell under earthquake. Based on these observations, two damper installation schemes are presented to determine the effective damper placement for satisfactory control performance. For vibration control, the equation of motion of controlled reticulated shell is deduced following the finite element theory. The local feedback control strategy with a Kalman filter proposed in Chapter 6 for the shear building can hardly be applied to the reticulated
shell because of the large number of accelerometers required and the difficulties involved in solving the Riccati Equation for the Kalman filter process. Therefore, a local control strategy using only the local information of friction dampers is utilized to realize the seismic mitigation. For system identification, the parameter identification scheme based on adding known stiffness proposed in Chapter 6 is extended to that for the reticulated shell. The transform matrix of stiffness parameters are deduced based on the structural connectivity and transformation information. In addition, the damage detection method proposed for shear building in Chapter 6 is also extended to that for the reticulated shell.

The numerical results demonstrate that the dynamic responses of the reticulated shell can be effectively suppressed using the semi-active friction dampers based on the local control strategy with fixed increment of slipping force. The damper installation scheme No.2 presents better control performance than that of scheme No.1 and the scheme No.2 therefore is adopted to provide known stiffness for parameter identification and damage detection. The numerical results from parameter identification and damage detection clearly demonstrate that the identification results are accurate without noise contamination. The identification accuracy reduces rapidly with the increase of noise intensity. The stiffness parameters and damage events can be identified only under noise intensity no more than 0.1%. It is also found that for the reticulated shell, the optimum $SR$ for seismic response control is slightly larger than that for system identification, and hence the optimal $SR$ for response control is adopted for the integrated system. The numerical results demonstrate that the quality of parameter identification and damage detection for the reticulated shell is inferior to that of shear building under the same noise intensity.
The configuration of semi-active friction dampers determined in this chapter will be adopted in Chapter 9 for the design of the integrated system. In addition, the control algorithm, parameter identification algorithm and damage detection method presented in this chapter for the reticulated shell will also be utilized in Chapter 9 for the realization of integrated health monitoring and vibration control system.
Table 7.1 The VRFs of axial forces in the members within the first three circles

<table>
<thead>
<tr>
<th>Member No.</th>
<th>Radial members</th>
<th>Circular members</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Scheme No.1</td>
<td>Scheme No.2</td>
</tr>
<tr>
<td>1</td>
<td>30.38%</td>
<td>31.44%</td>
</tr>
<tr>
<td>2</td>
<td>29.94%</td>
<td>27.65%</td>
</tr>
<tr>
<td>3</td>
<td>34.38%</td>
<td>31.33%</td>
</tr>
<tr>
<td>7</td>
<td>30.14%</td>
<td>32.02%</td>
</tr>
<tr>
<td>8</td>
<td>31.40%</td>
<td>29.35%</td>
</tr>
<tr>
<td>9</td>
<td>36.84%</td>
<td>35.43%</td>
</tr>
<tr>
<td>13</td>
<td>29.98%</td>
<td>31.98%</td>
</tr>
<tr>
<td>14</td>
<td>31.89%</td>
<td>30.07%</td>
</tr>
<tr>
<td>15</td>
<td>38.96%</td>
<td>36.74%</td>
</tr>
<tr>
<td>19</td>
<td>30.12%</td>
<td>31.97%</td>
</tr>
<tr>
<td>20</td>
<td>31.50%</td>
<td>29.81%</td>
</tr>
<tr>
<td>21</td>
<td>38.18%</td>
<td>35.21%</td>
</tr>
<tr>
<td>25</td>
<td>30.39%</td>
<td>32.19%</td>
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<td>26</td>
<td>30.84%</td>
<td>29.09%</td>
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<td>27</td>
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<td>31</td>
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<td>32</td>
<td>29.77%</td>
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<td>36.08%</td>
<td>32.00%</td>
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<td>43</td>
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<td>45</td>
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<td>30.70%</td>
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<table>
<thead>
<tr>
<th>Member No.</th>
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<tbody>
<tr>
<td></td>
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<td>218</td>
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</tr>
<tr>
<td>219</td>
<td>-3.23%</td>
</tr>
<tr>
<td>220</td>
<td>-0.60%</td>
</tr>
<tr>
<td>221</td>
<td>8.530%</td>
</tr>
<tr>
<td>222</td>
<td>36.81%</td>
</tr>
<tr>
<td>223</td>
<td>16.44%</td>
</tr>
<tr>
<td>224</td>
<td>-2.36%</td>
</tr>
<tr>
<td>225</td>
<td>-1.58%</td>
</tr>
</tbody>
</table>
Table 7.2 Identification errors of stiffness parameters at 0.1% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1%</td>
<td>58</td>
<td>14.22</td>
</tr>
<tr>
<td>1%~5%</td>
<td>108</td>
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<tr>
<td>5%~10%</td>
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<td>21.82</td>
</tr>
<tr>
<td>20%~30%</td>
<td>40</td>
<td>9.81</td>
</tr>
<tr>
<td>30%~50%</td>
<td>15</td>
<td>3.67</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 7.3 Identification errors of stiffness parameters at 0.5% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1%</td>
<td>45</td>
<td>11.03</td>
</tr>
<tr>
<td>1%~5%</td>
<td>84</td>
<td>20.59</td>
</tr>
<tr>
<td>5%~10%</td>
<td>60</td>
<td>14.71</td>
</tr>
<tr>
<td>10%~20%</td>
<td>115</td>
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<td>30%~50%</td>
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<td>11.03</td>
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<tr>
<td>&gt;50%</td>
<td>10</td>
<td>2.45</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 7.4 Identification errors of stiffness parameters at 1.0% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1%</td>
<td>26</td>
<td>6.37</td>
</tr>
<tr>
<td>1%~5%</td>
<td>61</td>
<td>14.95</td>
</tr>
<tr>
<td>5%~10%</td>
<td>71</td>
<td>17.40</td>
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<tr>
<td>10%~20%</td>
<td>67</td>
<td>16.42</td>
</tr>
<tr>
<td>20%~30%</td>
<td>72</td>
<td>17.65</td>
</tr>
<tr>
<td>30%~50%</td>
<td>63</td>
<td>15.45</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>48</td>
<td><strong>11.76</strong></td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure 7.1 Nodes and element numbers of the reticulated shell:
(a) Node number ; (b) Element number.
Figure 7.2 Maximum axial forces and axial stresses under El Centro earthquake:
(a) Axial force (kN); (b) Axial stress (MPa).
Figure 7.3 Maximum bending moments under El Centro earthquake:
(c) Moment $M_y$ (kN.m); (d) Moment $M_z$ (kN.m).
Figure 7.4 Maximum nodal displacement under the El Centro earthquake.
Figure 7.5 Connection between semi-active friction damper and nodes.

Figure 7.6 Two installation schemes of semi-active friction dampers:
(a) Scheme No.1; (b) Scheme No.2.
Figure 7.7 Nodal VRFs under the El Centro earthquake.
Figure 7.8 Comparison of displacement response time history of node 1.

Figure 7.9 Variation of $VRF_s$ with $SR$.

7-39
Figure 7.10 Identification accuracy of stiffness parameters without noise contamination.

Figure 7.11 Damage detection of scenario 1 without noise contamination.
Figure 7.12 Damage detection of scenario 2 without noise contamination.

Figure 7.13 Damage detection of scenario 2 under 0.1% noise contamination.
CHAPTER 8

INTEGRATED HEALTH MONITORING AND VIBRATION CONTROL IN TIME DOMAIN

8.1 INTRODUCTION

The integrated health monitoring and vibration control procedure using semi-active friction dampers has been developed and applied to the shear building and the reticulated shell in Chapters 6 and 7 respectively. The structural parameters are identified based on adding known stiffness using semi-active friction dampers to create the two states for the structures: (1) the structures without any additional stiffness (clamping force is set at zero); and (2) the structures with additional stiffness (damper is in sticking state). The acceleration responses of the structures under these two states are measured and analyzed respectively through modal identification techniques to obtain the natural frequencies and mode shapes. The proposed identification approach for stiffness parameters is then developed utilizing the natural frequencies and mode shapes without or with adding known stiffness manipulated by semi-active friction dampers. In addition, the location and severity of damage events are determined based on the parameters of undamaged and damaged structures. A local control strategy is applied for seismic mitigation of the reticulated shell based on the measured control forces of semi-active friction dampers with the aiding of the force transducers. It is understood that the parameter identification is carried out based on the model parameters identified through two steps: measurement of acceleration responses and identification of modal parameters. In addition, the
parameter identification also requires two steps: clamping force is set at zero to create the structures with no additional stiffness and damper is in sticking state to provide the structures with additional stiffness. Practically, the development of identification approach directly based on the obtained structural responses may be a better way to simplify the identification process. Furthermore, for the structures installed with other kinds of semi-active or active control devices, the identification approach proposed in Chapters 6 and 7 cannot be directly applied because these control devices cannot provide the required two states of additional stiffness like the semi-active friction damper. In addition, for those structures under active control, the active control devices can provide control forces and activate the structures without other external excitations. Therefore, how to conduct the parameter identification and damage detection for the structures with other types of semi-active and active control devices is a practical issue. Actually for control devices which cannot provide the required two states of additional stiffness to a structure like the semi-active friction dampers, the parameter identification and damage detection of the controlled structure can be performed in the time domain as long as the control forces can be measured.

The system identification methods in the time domain have been studied in recent years and many publications in this field can be referred to. Toki et al (1989) proposed a time domain identification technique by which structural parameters and ground motion of an earthquake-excited structure could be identified using the measured structural responses only. Wang and Haldar (1994) developed an iterative least-squares method to identify simultaneously the structural parameters and ground motion of an earthquake excited structure. Wang and Haldar (1997) also extended
their identification method to the structures with limited observations. Majumder and Manohar (2003) proposed a time domain approach for damage detection in bridge structures using ambient vibration data. Chen and Li (2004) suggested a new time domain method for simultaneous identification of the structural parameters and the time history of the input excitation using output-only measurements. Ling and Haldar (2004) proposed a system identification procedure for nondestructive damage detection of structures, which is a finite element based time domain linear system identification technique capable of identifying structures at element level. Zhao and Xu et al (2006) proposed a hybrid identification method for the multi-storey buildings with unknown ground motion. After all the structural parameters are identified, the unknown earthquake-induced ground motion is constructed by solving a first-order differentiation equation. Numerical example demonstrates the feasibility and efficiency of the hybrid identification method.

This chapter studies the parameter identification and damage detection of structures with control devices aiming to examine the feasibility of applying the integrated monitoring and control procedure for civil engineering structures with control forces in the time domain. The vibration control using semi-active friction dampers has been conducted in Chapters 6 and 7, which will not be mentioned in this chapter. The equation of motion of the controlled structure is converted to the parameter identification equation when the inertia forces, damping forces, and restoring forces are linear functions of structural parameters. By taking control forces as known external forces together with measured structural responses, the least-squares method together with an amplitude-selective filter is then used to solve the parameter identification equation, from which the structural parameters can be identified. The
same procedure is applied to the controlled structure with damage to identify another set of structural parameters. By comparing the two sets of structural parameters identified, the structural damage can finally be detected and quantified. This proposed procedure is applied to the shear building and the reticulated steel shell with control devices for parameter identification and damage detection with and without measurement noise.

8.2 PARAMETER IDENTIFICATION

The objective of system identification in the time domain for controlled structures is to determine the structural parameters based on the measured dynamic responses, external excitations and control forces. The equation of motion for a structure with control devices under external excitations can be written as

\[ M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = R(t) + H^T u(t) \]  

(8.1)

where \(M\), \(C\) and \(K\) are the mass, damping and stiffness matrices of structure respectively; \(x(t)\), \(\dot{x}(t)\) and \(\ddot{x}(t)\) are the displacement, velocity and acceleration responses relative to the ground respectively; \(R(t)\) is the external excitations; \(u(t)\) is the control forces provided by the control devices; \(H^T\) is the influence matrix reflecting location of control forces.

The Rayleigh damping assumption is adopted to construct the structural damping matrix (Clough and Penzien 1975)

\[ C = \alpha M + \beta K \]  

(8.2)

where \(\alpha\) is the mass damping coefficient; \(\beta\) is the stiffness damping coefficient.

Substitute Equation (8.2) into Equation (8.1) yields

\[ M\ddot{x}(t) + \alpha M\dot{x}(t) + \beta K\dot{x}(t) + Kx(t) = R(t) + H^T u(t) \]  

(8.3)
Equation (8.3) can be rewritten as

\[ f_I(t) + f_{DM}(t) + f_E(t) = R(t) + H'u(t) \]  

(8.4)

where

\[ f_I(t) = M\ddot{x}(t) \]  

(8.5)

\[ f_{DM}(t) = \alpha M\ddot{x}(t) \]  

(8.6)

\[ f_E(t) = K(\beta\ddot{x}(t) + x(t)) \]  

(8.7)

For the shear building investigated in this study, the stiffness parameter is selected as the floor stiffness coefficient. For the reticulated shell investigated in this study, the stiffness parameter is selected as the product of Young’s modulus and cross section area (EA) of each member. These stiffness parameters can be extracted from elastic forces based on response vector sensitivities (Zhao and Xu 2006). Therefore, the elastic force is the linear function of stiffness parameters and the elastic force can be expressed as

\[ f_E(t) = E(t) \gamma(t) \gamma(t) \]  

(8.8)

\[ f_E(t) = H(t)\theta \]  

(8.9)

where

\[ H(t) = \begin{bmatrix} \frac{\partial f_E(t)}{\partial \gamma(1)} & \frac{\partial f_E(t)}{\partial \gamma(2)} & \ldots & \frac{\partial f_E(t)}{\partial \gamma(ne)} \end{bmatrix} \]  

(8.10)

\[ \theta = (\gamma(1), \gamma(2), \ldots, \gamma(ne))' \]  

(8.11)

Assuming that the mass parameters and damping coefficients are known, Equation (8.4) can be rewritten as

\[ H(t)\theta = z(t) \]  

(8.12)

where

\[ z(t) = R(t) + H'u(t) - f_I(t) - f_{DM}(t) \]  

(8.13)

in which \( ne \) is the total number of elements of the structure; \( \gamma(m) \) is the stiffness parameter of the \( m \)th element.

The derivative of \( f_E(t) \) to the stiffness parameter \( \gamma(m) \) of the \( m \)th element is

\[ \frac{\partial f_E(t)}{\partial \gamma(m)} = \frac{\partial K}{\partial \gamma(m)} (\beta\ddot{x}(t) + x(t)) \]  

(8.14)
The sensitivity matrix of the global stiffness matrix \( K \) to the stiffness parameter of the \( m \)th element can be written as

\[
S_{k}^{(m)} = \frac{\partial K}{\partial \gamma^{(m)}} = \frac{\partial K_{e}^{(m)}}{\partial \gamma^{(m)}} = \left[ T^{(m)} \right]^T \frac{\partial K_{e}^{(m)}}{\partial \gamma^{(m)}} = \left[ T^{(m)} \right]^T S_{k,e}^{(m)} T^{(m)} \quad (8.15)
\]

where \( K_{e}^{(m)} \) is the element stiffness matrix of the \( m \)th element in the local coordinate system; \( K^{(m)} \) is the contribution matrix of the element stiffness matrix of the \( m \)th element \( K_{e}^{(m)} \) to the global stiffness matrix \( K \); \( S_{k}^{(m)} \) is the sensitivity matrix of the global stiffness matrix to the \( m \)th stiffness parameter \( \gamma^{(m)} \); \( S_{k,e}^{(m)} \) is the sensitivity matrix of the element stiffness matrix \( K_{e}^{(m)} \) to the \( m \)th stiffness parameter \( \gamma^{(m)} \); \( T^{(m)} \) is the freedom transform matrix from local coordinate system to global coordinate system, which is the products of coordinate transformation matrix \( T_{a}^{(m)} \) and position matrix \( T_{c}^{(m)} \) of the \( m \)th element

\[
T^{(m)} = T_{a}^{(m)} T_{c}^{(m)} \quad (8.16)
\]

Therefore, the derivative of \( f_{e}(t) \) to the stiffness parameter \( \gamma^{(m)} \) of the \( m \)th element

\[
\frac{\partial f_{e}(t)}{\partial \gamma^{(m)}}
\]

can be written as

\[
s_{t_{e}}^{(n)}(t) = \frac{\partial f_{e}(t)}{\partial \gamma^{(m)}} = S_{k}^{(m)} (\beta \hat{x}(t) + x(t))
\quad (8.17)
\]

Equation (8.10) can be expressed as

\[
H(t) = \left[ s_{t_{e}}^{(1)}(t), s_{t_{e}}^{(2)}(t), ..., s_{t_{e}}^{(n)}(t) \right]
\quad (8.18)
\]

Equation (8.12) is the dynamic balance equation of the controlled structure which is also the identification equation at time instant \( t \). The coefficient matrix \( H(t) \) is constructed using the sensitivity matrix of \( f_{e}(t) \) to the stiffness parameter \( \theta \) and the vector \( z(t) \) is constructed based on known external excitations \( R(t) \), known control
forces $u(t)$, $f_i(t)$ and $f_{DM}(t)$ at time instant $t$. Assembling the identification equations (See Equation (8.12)) at all sampling instants results in the final identification equation for the stiffness parameters

$$\mathbf{H}\theta = \mathbf{z} \quad (8.19)$$

The flow chart of identification process for stiffness parameters in the time domain is plotted in Figure 8.1.

### 8.3 Damage Detection

Despite having installed with control devices, the structure may still suffer some damage after extreme events or long-term service. In this section, the identification scheme is applied to the damaged structure for the identification of the stiffness parameters. By comparing with the stiffness parameters of the undamaged and damaged structures, the location and severity of the damage events can be determined. For simplicity, the mass and damping matrices of the damaged structure are assumed the same as those of the original structure. Similar to the undamaged structure, the identification equation of stiffness parameter at a certain time instant can be written as

$$\mathbf{H}^d(t)\theta^d = \mathbf{z}^d(t) \quad (8.20)$$

where

- $\mathbf{H}^d(t) = \left[ s^{d(1)}_i(t), s^{d(2)}_i(t), ..., s^{d(ne)}_i(t) \right] \quad (8.21)$
- $\theta^d = (\gamma^{d(1)}, \gamma^{d(2)}, ..., \gamma^{d(ne)})^T \quad (8.22)$
- $\mathbf{z}^d(t) = \mathbf{R}(t) + \mathbf{H}'u'(t) - f^d_i(t) - f_{DM}^d(t) \quad (8.23)$
- $f^d_i(t) = \mathbf{M}\dot{x}^d(t) \quad (8.24)$
- $f_{DM}^d(t) = \alpha\mathbf{M}\dot{x}^d(t) \quad (8.25)$
- $f_{\alpha}^d(t) = \mathbf{K}^d(\beta\dot{x}^d(t) + \dot{x}^d(t)) \quad (8.26)$
in which \( ne \) is the total number of elements of the structure; \( \gamma^{d(m)} \) is the stiffness parameter of the \( m \)th element of the damaged structure; \( \mathbf{M} \) and \( \mathbf{C} \) are the mass and damping matrices of damaged structure (same as undamaged structure); \( \mathbf{K}^d \) is the stiffness matrix of damaged structure; \( \mathbf{x}^d(t) \), \( \mathbf{\dot{x}}^d(t) \) and \( \mathbf{\ddot{x}}^d(t) \) are the displacement, velocity and acceleration responses of damaged structure relative to the ground under known excitation respectively; \( \mathbf{R}(t) \) is the known excitations; \( \mathbf{u}^d(t) \) is the control forces provided by the control devices, which can be measured through force transducers; \( \mathbf{H}^e \) is the influence matrix reflecting the location of control forces; \( \alpha \) and \( \beta \) are the mass and stiffness damping coefficients respectively.

Assembling the identification equations (See Equation (8.20)) at all sampling instants would result in the final identification equation for the stiffness parameters of the damaged structure

\[
\mathbf{H}^e \theta^d = \mathbf{z}^d
\]  

(8.28)

Once the stiffness parameters of the damaged structure are found, the damage location and severity can be determined by analyzing the results from the following equation.

\[
\Delta \theta = \theta^d - \theta
\]  

(8.29)

### 8.4 Influence of Unknown Excitation

For the structures protected by the semi-active control devices, the structural stiffness parameters can be identified by using the measured information of input excitation and control forces. This identification process has been presented in the former
sections. If the structures are installed with active control devices, these active devices can produce control forces to activate the structures. Under this circumstance, the structures vibrate under the excitations produced by active control devices. If the control forces provided by the active control devices to activate the structures cannot be fully measured, the identification of the structural stiffness parameters may be affected to some extent. It is therefore necessary to examine the influence of unknown excitations produced by control devices on the identification performance.

If the structure vibrates only under the active control forces, the identification equation (8.19) can be written by setting external excitation $R(t)$ as zero

$$
\begin{bmatrix}
H_k \\
H_u
\end{bmatrix}
\theta =
\begin{bmatrix}
z_k \\
z_u
\end{bmatrix}
$$

$$
z_k = H_k^* u_k - f_k^I - f^I_{DM}
$$

$$
z_u = H_u^* u_u - f_u^I - f^I_{DM}
$$

where $u_k$ and $u_u$ are the known and unknown active control forces respectively; $H_k$ is the influence matrix corresponding to $u_k$; $H_u$ is the influence matrix corresponding to $u_u$; $f_k^I$ and $f_u^I$ are the force vectors of $f_I$ corresponding to the known and unknown active control forces $u_k$ and $u_u$ respectively; $f^I_{DM}$ and $f^I_{DM}$ are the force vectors of $f_{DM}$ corresponding to the known and unknown active control forces $u_k$ and $u_u$ respectively; $z_k$ and $z_u$ are the known and unknown vectors of $z$ respectively;

Considering Equations (8.30) to (8.32), the identification equation including unknown force $z_u$ can be depicted using two uncoupled equations

$$
H_k \theta = H_k^* u_k - f_k^I - f^I_{DM}
$$

$$
H_u \theta = H_u^* u_u - f_u^I - f^I_{DM}
$$

8-9
Obviously, the stiffness parameters can be determined through solving the identification equation (8.33) and the detection accuracy greatly depends on the mathematical characteristics of matrix $H_k$. If $H_k$ is full column rank, Equation (8.33) does not describe an underdetermining problem and the least-squares solution is

$$\theta = H_k^+ (H_k^T u - f_k^T - f_{DM}^T) \quad (8.35)$$

If the number of unknown active control forces is quite large, $H_k$ may have many columns with zero vectors. In this circumstance, the solution of Equation (8.35) cannot be directly obtained because the pseudo inverse $H_k^+$ does not exist. In this regard, $H_k$ can be divided as

$$H_k = [H_{kk} \ 0] \quad (8.36)$$

in which $H_{kk}$ is a sub-matrix of $H_k$ with full column rank.

Similarly, the stiffness parameter $\theta$ can also be divided as

$$\theta = \begin{bmatrix} \theta_k \\ \theta_u \end{bmatrix} \quad (8.37)$$

where $\theta_k$ is the stiffness parameter corresponding to the non-zero coefficient matrix $H_{kk}$; $\theta_u$ is the stiffness parameter corresponding to the zero parts of coefficient matrix $H_k$.

Substituting Equations (8.36) and (8.37) into Equation (8.33) yields

$$\begin{bmatrix} H_{kk} & 0 \end{bmatrix} \begin{bmatrix} \theta_k \\ \theta_u \end{bmatrix} = H_k^T u - f_k^T - f_{DM}^T \quad (8.38)$$

Owing to the full column rank of coefficient matrix $H_{kk}$, the parameter $\theta_k$ can be determined

$$\theta_k = H_{kk}^+ (H_k^T u - f_k^T - f_{DM}^T) \quad (8.39)$$

Dividing the coefficient $H_u$ in the similar way adopted in Equation (8.34) yields
Rewrite Equation (8.40) as

\[
\begin{bmatrix}
H_{ik} & H_{iu}
\end{bmatrix} \begin{bmatrix}
\theta_k \\
\theta_i
\end{bmatrix} = \begin{bmatrix}
H_u \nu - f_i - f_{DM}
\end{bmatrix}
\]  

(8.41)

Substituting Equation (8.39) into Equation (8.41) yields

\[
H_{ik} \nu - f_i - f_{DM} - H_{iu} \theta_u = H_{ik} \begin{bmatrix}
H_u \nu - f_i - f_{DM}
\end{bmatrix}
\]  

(8.42)

If the number of unknown active control forces is quite large, the coefficient matrix \( H_k \) shall have many zero columns. In this situation, only partial stiffness parameter \( \theta_k \) can be identified. The unknown stiffness parameters \( \theta_u \) and the unknown active control forces \( u_u \) satisfy the relationship in Equation (8.42). It is understood that if all the active control forces can be measured, the structural stiffness parameters can be effectively identified. If the structure is subjected to unknown active control forces, the identification results of stiffness parameters depend on the number and location of unknown active control forces.

8.5 NUMERICAL INVESTIGATION ON PARAMETER IDENTIFICATION

8.5.1 Parameter identification of shear building

8.5.1.1 Description of shear building

The five-storey shear building adopted in Chapter 6 is firstly selected as an example building to discuss the identification quality. The example building has the same storey height of 3m. The mass and the horizontal storey (shear) stiffness of the original building without damage are uniform for all storeys with mass \( m = 5.1 \times 10^5 \text{ kg} \) and stiffness \( k = 1.334 \times 10^7 \text{ N/m} \). The equation of motion of the building is solved using the Newmark-\( \beta \) method with a time step of 0.002s. The two
factors in the Newmark-\( \beta \) method are selected as \( \alpha = 1/2 \) and \( \beta = 1/4 \). The Rayleigh damping assumption is adopted to construct the structural damping matrix. The damping ratios in the first two modes of vibration of the building are assumed to be 0.02. A semi-active friction damper with a diagonal brace is used to connect two neighboring floors. The same arrangement with the same damper is made for each storey of the building (See Figure 8.2). The excitation is assumed to be a white noise random process with a peak value of 5.4\( kN \) (See Figure 8.3) acting on the first floor of the building. The sampling frequency of 500Hz is used in the simulation. The displacement, velocity and acceleration responses of all the floors are assumed to be measured for parameter identification. In the case of parameter identification, the clamping forces of the semi-active friction dampers are set at a certain value to produce the slipping force of 500\( N \) when the building is subjected to the excitation shown in Figure 8.3. The external excitation and control forces of all the semi-active friction dampers are assumed to be measured in the identification process. The stiffness ratio of the additional stiffness from the brace to the horizontal stiffness of the building storey defined and adopted in Chapter 6 is selected as 0.9.

8.5.1.2 Identification of stiffness parameters

For the five-storey shear building concerned, the horizontal storey stiffness coefficients of all the floors are required to be identified

\[
\theta = [k_1, k_2, k_3, k_4, k_5]^T
\]

(8.43)

The structural dynamic responses are computed and displayed in Figure 8.4 for the time histories of displacement, velocity, and acceleration responses at the top floor of the building. Displayed in Figure 8.5 are the time histories of control forces for all the five semi-active friction dampers. By using the measured information of dynamic
responses, control forces and external excitation without considering noise contamination, the exact stiffness coefficients of the building can be identified and the identified results are shown in Figure 8.6. It can be seen that the exact values of five horizontal storey stiffness coefficients can be accurately identified without noise contamination. In practice, the noise contamination is unavoidable in the measured dynamic acceleration responses. Therefore, the effects of measurement noise on the identification quality of stiffness parameters should be examined. The measurement noise, which is assumed to be a white noise random process, is simulated and added to the dynamic responses, control forces and external excitation according to a given noise intensity. The noise intensity is defined as the ratio of the root mean square (RMS) of the noise to the RMS of the time history of original signal

\[
\text{Noise intensity} = \frac{\text{RMS(noise)}}{\text{RMS(signal)}} \times 100\% \quad (8.44)
\]

Because the measurement noise considered in this study is assumed to be normally distributed white noise, the amplitude of measurement noise at one frequency is the same as that at the other frequency in the frequency domain. In the time domain, the amplitude of measurement noise at most time instants is below a certain value. For instance, for a normally distributed white noise the noise amplitude at 95\% of the sampling points of measurement noise time history is below 1.645 times its RMS. Since the amplitude of structural response at each time instant is different, the relative error induced by measurement noise would be different from time instant to time instant. According to the noise intensity defined in Equation (8.44), the relative noise level will be small for the structural response of large amplitude in general. Furthermore, it is not necessary to take all the time instants (the sampling points) in one time history into full consideration when using the identification method. Therefore based on the above observations, Zhao and Xu et al (2006) proposed an
amplitude-selective filtering procedure to filter the structural responses below a preset threshold. Only the structural responses above the given threshold are retained in the identification process so as to reduce the effect of measurement noise and at the same time to improve the identification quality. The criterion used to determine the threshold is the number of the retained sampling points of structural responses. The number of retained sampling points should be smaller compared with the total number of sampling points, but it should be larger enough to satisfy the effective solution to identification equation. For a single-response time history \( x_i \) of dimension \( n \), if the number of retained sampling points is set to \( m \) the retained structural response time history after the filtering can be expressed as

\[
x_j = \{x_i \mid x_i > x^*\}
\]  

(8.45)

where \( x^* \) is the \((n-m)\) largest value in the original time history \( x_i \). For more than one structural response time history, in consideration that different types of structural responses are involved and the same noise level is assigned to each structural response, the sampling points are discarded one by one starting from the point with the smallest response until the retained number reaches the given number but this procedure should be applied to all the response time histories with an even chance.

Three noise intensities, 0.5\%, 1.0\%, and 2\%, are introduced to the structural dynamic responses, control forces of semi-active friction dampers and external excitation to assess the effects of noise contamination on the identification quality of the stiffness parameters. The retained number of sampling points is set at 100 following Equation (8.45). Obviously from Figure 8.7, the identification errors gradually increase with the increasing noise intensity and the maximum relative identification error in the stiffness coefficients is about 1.6\% under 2\% noise intensity. The above
identification observations are made based on the white noise excitation acting on the first floors. Actually, the identification cases with other kinds of excitations such as sinusoidal excitation acting on other floors of the shear building are also analyzed. Similar observations can be made to those reported in this section.

For the semi-active friction dampers adopted in this thesis for health monitoring and vibration control, the clamping forces of the semi-active friction dampers can be set at zero level to create the status of the building without any control forces. The stiffness parameters of the shear building under this damper state are also identified for performance comparison. The adopted external excitation is the same as that used in Section 8.5.1.1. The displacement, velocity and acceleration responses of the five-storey shear building are utilized in the parameter identification. The retained number of sampling points is set at 100. Displayed in Figure 8.8 are the time histories of displacement, velocity, and acceleration responses at the top floor of the building without control forces. By using all the three kinds of dynamic responses without considering noise contamination, the exact stiffness coefficients of the building can be identified and the identified results are shown in Figure 8.9. It can be seen that the exact values of five horizontal storey stiffness coefficients can be accurately identified without noise contamination. Plotted in Figure 8.10 are the relative identification errors in the stiffness coefficients at the three levels of noise. It can be seen from Figure 8.10 that identification errors increase with the increasing noise intensity. The maximum relative identification error in the stiffness coefficients is about 1% for 2% noise intensity.
By comparing the identification quality with/without control forces, one can understand that both of them can accurately determine the structural stiffness parameters without noise contamination. With the introduction of noise contamination, the identification quality without control forces is slightly better than that with control forces. This is because the measured time histories of control forces of the semi-active friction dampers utilized in the identification process are also polluted by the noise contamination. The introduction of more contaminated signals may slightly reduce the identification quality.

8.5.2 Parameter identification of reticulated shell

8.5.2.1 Description of reticulated shell

The parameter identification by using semi-active friction dampers in the time domain is presented and applied to the shear building in the former section. This section will investigate the identification quality of the reticulated shell without/with noise contamination. As discussed in Chapter 7, the bending responses (bending moments and bending stresses) of shell members under dead loads and dynamic loads are quite small compared to axial responses (axial forces and axial stresses). The rigid connection of nodes can be simplified as joint connection. The structural performance of the reticulated shell with rigid connection is also similar to the reticulated shell with the same geometric dimension and physical parameters but with joint connection. In the parameter identification of the reticulated shell discussed in Chapter 7, the original rigid connection of the nodes is simplified as joint connection to simplify the analytical model. Only the axial stiffness parameters of the shell members (EA) are identified. The damper installation scheme No.2 with satisfactory control performance is adopted in the parameter identification and
damage detection. Therefore, the simplified shell model with damper installation scheme No.2 is also adopted in this chapter to examine the performance of parameter identification and damage detection in the time domain. The numbers of nodes and elements of the reticulated shell are displayed in Figure 8.11.

The parameter identification without/with noise contamination is carried out in this section to estimate the stiffness parameters (EA) of the reticulated shell. As discussed in Chapter 7, the dynamic responses of nodes and members within the first three circles are larger than those at other places of the shell. The external excitation acting within this region can more effectively activate the reticulated shell. Therefore, the white noise excitation P1 adopted for the shear building (See Figure 8.3) is utilized for the reticulated shell and assumed to be added in vertical direction of node 23 as displayed in Figure 8.12. The equation of motion of the shell under excitation P1 is solved using the Newmark-\(\beta\) method with a time step of 0.002s. The two factors in the Newmark-\(\beta\) method are selected as \(\alpha = 1/2\) and \(\beta = 1/4\). The Rayleigh damping assumption is adopted to construct the structural damping matrix, and the damping ratios in the first two modes of vibration of the reticulated shell are set at 0.01. Similar to the parameter identification of the shear building, the clamping forces of the semi-active friction dampers are set at a certain value to produce the slipping force of 500N when the shell is subjected to white noise excitation P1. In addition, the clamping forces of the semi-active friction dampers are set at zero level to create the status of the reticulated shell without any control forces for performance comparison. The stiffness ratio of the semi-active damper is assumed 1.2 which is the same as that utilized in Chapter 7. In the identification process, the external excitation and control forces of all the semi-active friction dampers are assumed to
be known. In addition, the measured dynamic information is assumed to be completed which means that the displacement, velocity and acceleration responses of all the nodes are assumed to be known and utilized in the identification process.

8.5.2.2 Identification of stiffness parameters

By using displacement, velocity and acceleration responses without considering noise contamination, the exact stiffness parameters of the reticulated shell can be accurately identified with/without control forces as shown in Figure 8.13. The effects of measurement noise on the identification quality of stiffness parameters are also examined. The measurement noise, which is assumed to be a white noise random process, is simulated and added to the dynamic responses, control forces and external excitation according to a given noise intensity. The noise intensity is defined in Equation (8.44) and three noise intensities, 0.1%, 0.5%, and 1%, are introduced to the original signals. The retained number of sampling points is set at 100 following amplitude-selective filtering procedure expressed in Equation (8.45).

The identification errors of stiffness parameters with known control forces under 0.1%, 0.5% and 1% noise intensities are computed and listed in Tables 8.1, 8.2, and 8.3 respectively. The results in Table 8.1 demonstrate that all of the stiffness parameters can be effectively identified under 0.1% noise contamination because the identification errors of 99.5% of all the members are no more than 5%. The identification errors of stiffness parameters under 0.5% noise intensity listed in Table 8.2 reveal that the identification errors of 78% of all the members are no more than 10%. While the identification errors listed in Table 8.3 for 1% noise contamination demonstrate that about 8% of all the members have the identification errors no more
than 10%. Obviously, the identification quality of stiffness parameters under 1.0% noise intensity is not as good as that under 0.1% and 0.5% noise intensities. Similarly, the identification errors of stiffness parameters without control forces under 0.1%, 0.5% and 1% noise intensities are computed and listed in Tables 8.4, 8.5, and 8.6 respectively. The results in Table 8.4 demonstrate that all of the stiffness parameters can be effectively identified under 0.1% noise contamination because the maximum identification error is no more than 5%. The identification errors of stiffness parameters under 0.5% noise intensity listed in Table 8.5 are no more than 20%. The identification errors listed in Table 8.6 for 1% noise contamination demonstrate that about 57% of all the members have the identification errors no more than 10%.

By summarizing the identification results of the reticulated shell, one can find that all the stiffness parameters can be accurately identified by using the presented identification methods with/without control forces under noise free situation. Under noise contamination, the identification quality without control forces is better than that with the control forces. This is because the utilized information of control forces of semi-active friction dampers are polluted by the noise contamination. The introduction of more contaminated signals may reduce the identification quality.

8.6 NUMERICAL INVESTIGATION ON DAMAGE DETECTION

8.6.1 Damage detection of shear building

The feasibility of the damage detection scheme discussed in this chapter for identifying different damage severities and locations is examined in this section. The effects of noise contamination on the damage detection quality are also assessed.
Two damage scenarios are taken into consideration in this section, which are the same as those adopted in Chapter 6: (1) single damage event with 20%, 30%, 40% and 50% stiffness loss, respectively, at the first storey of the example building; and (2) double damage event with 20% and 30% stiffness loss at the first and fourth storey respectively. The two levels of measurement noise of 1% and 2% are considered in the damage detection. The external excitation and the slipping force of the semi-active friction damper adopted are the same as those used in the parameter identification process. In addition, the damage detection of the structures without control forces is also carried out for comparison. The dynamic responses of all the floors to the input external excitation are computed. The measurement noise is simulated and added to the dynamic responses according to a given noise intensity. The retained number of sampling points is set at 100 following amplitude-selective filtering procedure.

The damage size defined in Chapter 7 is computed for the single damage event in the first storey, and the results are displayed in Figures 8.14 (a) to (d) for 20%, 30%, 40% and 50% stiffness loss cases with measured control forces, respectively. It can be seen that without measurement noise, the proposed detection scheme can accurately determine the damage location and size. With measurement noise involved, the proposed detection scheme can still determine the damage location and size satisfactorily. With the increase of measurement noise intensity, the damage size detected is less accurate. Displayed in Figure 8.15 are the damage sizes for the double damage event. It can be seen that without measurement noise, the proposed detection scheme can exactly determine the damage location and size. With measurement noise considered, the damage location and size can still be effectively
determined but the accuracy of damage detection is reduced with the increase of measurement noise intensity. The damage sizes for the single and double damage events without control forces are also computed and the results are displayed in Figures 8.16 and 8.17 respectively. Similar conclusions can be reached to those obtained with known control forces. The comparison of the identification quality between the cases with/without control forces demonstrates that both of them can accurately determine the structural stiffness parameters without noise contamination. With the introduction of noise contamination, the identification quality without control forces is slightly better than those with control forces.

8.6.2 Damage detection of reticulated shell

The performance of the damage detection scheme for identifying damage events of the reticulated shell is examined in this section which includes the effects of noise contamination on the detection quality. The stiffness parameter is selected as the product of Young’s modulus and cross section area of member (EA). Two damage scenarios are taken into consideration in this section which is the same as those adopted in Chapter 7: (1) 20% axial stiffness loss at the member 1; (2) 20%, 30% and 40% axial stiffness loss at member 1, 7 and 49 respectively. The locations of damaged members are plotted in Figure 8.11. Three noise intensities, 0.1%, 0.5%, and 1% are considered in the damage detection. The known external excitation used is the same as that used in the parameter identification. Similar to the parameter identification of the shell, the clamping forces of the semi-active friction dampers are then set at a certain value to produce the slipping force of 500N. The damage detection based on the shell without control forces is also conducted for comparison. The dynamic responses of all the nodes to the input excitation are assumed to be
measured. The measurement noise is simulated and added to the dynamic responses, input excitation and control forces of semi-active friction dampers according to a given noise intensity. The retained number of sampling points is set at 100 following amplitude-selective filtering procedure. The stiffness ratio of the semi-active friction damper is adopted 1.2 which is the same as that utilized in the parameter identification.

The noise free damage sizes are computed for the damage events with control forces and the results are displayed in Figures 8.18 (a) and (b) for damage scenarios 1 and 2 respectively. It can be seen that without measurement noise, the detection scheme can accurately determine the damage location and size. The same observations can also be made from the detection results without control forces under noise free situation. The damage scenario 2 with three damaged members is adopted to examine the effects of noise contamination on detection quality. Displayed in Figure 8.19 are the detection results of the first 50 members for the damage scenario 2 with/without control forces under 0.1% noise contamination. It is seen that the damage location can be effectively determined by comparing the detected damage sizes of all the members. Obviously under 0.1% noise intensity, the damage location and size can still be effectively determined but the accuracy of damage sizes is reduced to some extent. The damage detection of the reticulated shell under 0.5% and 1% noise intensities are also carried out and the results demonstrate that the detection errors of some members are quite large and the damage location cannot be accurately determined. The comparison of the detection quality between the cases with/without control forces demonstrates that both of them can accurately determine the structural stiffness parameters without noise contamination. With the introduction of noise
contamination, the identification quality without control forces is slightly better than those with control forces.

8.7 SUMMARY

The parameter identification and damage detection of the controlled structures are numerically investigated in the time domain in this chapter. For control devices which cannot provide the required two states of additional stiffness to a structure like the semi-active friction dampers, the parameter identification and damage detection of the controlled structure can be performed in the time domain as long as the control forces can be measured. This procedure can also be applied to the structures installed with semi-active or active control devices besides semi-active friction dampers. The equation of motion of the controlled structure is first converted to the parameter identification equation when the inertia forces, damping forces, and restoring forces are linear functions of the structural parameters. By taking control forces as known external forces together with measured structural responses, the least-squares method adopting an amplitude-selective filter is then used to solve the parameter identification equation, from which the structural parameters can be identified. The same procedure is applied to the controlled structure with damage to identify another set of structural parameters. By comparing the two sets of structural parameters identified, the structural damage can finally be detected and quantified. This proposed procedure is applied to the shear building and the reticulated steel shell with control devices for parameter identification and damage detection with and without measurement noise.

In parameter identification and damage detection, the clamping forces of the semi-active friction dampers are set at a certain value to produce the slipping force of
500 N when the structures are subjected to the known excitation. In addition, the clamping forces of the semi-active friction dampers are set at zero level to create the status of the structures without control forces for performance comparison. The identification performance under these two cases is numerically examined and compared without/with noise contamination.

The identification process is firstly carried out based on the known excitations and control forces. The identification results demonstrate that all the stiffness parameters of the shear building and the reticulated shell can be accurately identified by utilizing noise free measurement information of dynamic responses. As far as the shear building is concerned, the identification errors gradually increase with the increasing noise intensity. The damage location and size of single and double damage events can be effectively detected even under 2% noise intensity. The identification accuracy of stiffness parameters for the reticulated shell, however, rapidly decreases with the introduction of noise contamination. The identification accuracy of stiffness parameters is satisfactory only under 0.1% noise intensity. The damage detection results of the reticulated shell demonstrate that the damage location and size can be identified satisfactorily only under 0.1% noise intensity. The identification process based on the known external excitations and zero control forces is also conducted to examine the identification quality. To compare the identification quality with/without control forces, one can find that both of them can accurately determine the stiffness parameters of the shear building and the reticulated shell without noise contamination. With the introduction of noise contamination, the identification quality without control forces is slightly better than those with known control forces. This is because the measured time histories of control forces of semi-active friction
dampers utilized in the identification process are also polluted by the noise contamination. The introduction of more contaminated signals shall slightly reduce the identification quality. The numerical results also demonstrate that the quality of parameter identification and damage detection for the reticulated shell is inferior to that of shear building under the same noise intensity.

The performance comparison between the identification approaches in the frequency domain (in Chapters 6 and 7) and the time domain (in this chapter) demonstrates that the approach in the frequency domain can only be utilized to structures with semi-active friction dampers while the approach presented in this chapter can be applied to structures with other kinds of semi-active or active control devices. The stiffness parameters and the damage events of the shear building and the reticulated shell can be accurately identified without noise contamination by using the algorithm proposed in the frequency domain based on enough modal information. By using completed dynamic responses and known input excitations without noise contamination, the stiffness parameters and damage events of the shear building and the reticulated shell can also be accurately identified using the time domain approach presented in this chapter. The results of parameter identification and damage detection under noise contamination reveal that the identification quality in this chapter is a little bit better than that obtained in the former chapters. The observations made in parameter identification and damage detection of the reticulated shell in Chapter 7 and this chapter shall provide beneficial instructions for the design of the integrated health monitoring and vibration control system discussed in Chapter 9.
Table 8.1 Identification errors of stiffness parameters with control forces at 0.1% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1%</td>
<td>225</td>
<td>55.15</td>
</tr>
<tr>
<td>1%~5%</td>
<td>181</td>
<td>44.36</td>
</tr>
<tr>
<td>5%~10%</td>
<td>2</td>
<td>0.49</td>
</tr>
<tr>
<td>10%~20%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20%~30%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>30%~50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 8.2 Identification errors of stiffness parameters with control forces at 0.5% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
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<tbody>
<tr>
<td>&lt;1%</td>
<td>9</td>
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<tr>
<td>1%~5%</td>
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<td>24.51</td>
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<tr>
<td>5%~10%</td>
<td>203</td>
<td>49.75</td>
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<tr>
<td>10%~20%</td>
<td>92</td>
<td>22.54</td>
</tr>
<tr>
<td>20%~30%</td>
<td>3</td>
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<td>30%~50%</td>
<td>1</td>
<td>0.24</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 8.3 Identification errors of stiffness parameters with control forces at 1.0% noise level

<table>
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<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
<td>5%~10%</td>
<td>21</td>
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<td>31.12</td>
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<tr>
<td>20%~30%</td>
<td>167</td>
<td>40.93</td>
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<tr>
<td>30%~50%</td>
<td>77</td>
<td>18.87</td>
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<tr>
<td>&gt;50%</td>
<td>4</td>
<td>0.98</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
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</tr>
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</table>

Table 8.4 Identification errors of stiffness parameters without control forces at 0.1% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
</tr>
</thead>
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<tr>
<td>&lt;1%</td>
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</tr>
<tr>
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<td>32</td>
<td>7.84</td>
</tr>
<tr>
<td>5%~10%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10%~20%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>20%~30%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>30%~50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 8.5 Identification errors of stiffness parameters without control forces at 0.5% noise level

<table>
<thead>
<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
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<tr>
<td>&lt;1%</td>
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<tr>
<td>1%~5%</td>
<td>241</td>
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</tr>
<tr>
<td>5%~10%</td>
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<td>17.40</td>
</tr>
<tr>
<td>10%~20%</td>
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<td>2.20</td>
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<tr>
<td>20%~30%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>30%~50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>&gt;50%</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Sum</td>
<td>408</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 8.6 Identification errors of stiffness parameters without control forces at 1.0% noise level

<table>
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<tr>
<th>Error range</th>
<th>The number of members</th>
<th>Percent (%)</th>
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<td>&lt;1%</td>
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<td>100</td>
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<td>1.22</td>
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<td>0</td>
<td>0</td>
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<tr>
<td>Sum</td>
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</table>
Figure 8.1 Flow chart of identification process for stiffness parameters in time domain.
Figure 8.2 Five-storey shear building with semi-active friction dampers.

Figure 8.3 Time histories of external excitation.
Figure 8.4 Time histories of dynamic responses at the top floor.
Figure 8.5 Time histories of control forces.
Figure 8.6 Identification results of stiffness coefficients without noise contamination.

Figure 8.7 Identification results of stiffness coefficients with noise contamination.
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Figure 8.9 Identification results of stiffness coefficients without noise contamination and control.
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Figure 8.12 Position of known external excitation.

Figure 8.13 Identification accuracy of the reticulated shell without noise contamination: (a) With control forces; (b) Without control forces.
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Figure 8.15 Damage detection results for double damage event with control forces.
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Figure 8.17 Damage detection results for double damage event without control forces.
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Figure 8.19 Damage detection of scenario 2 with noise contamination.
CHAPTER 9

DESIGN OF INTEGRATED MONITORING AND
CONTROL SYSTEM FOR RETICULATED SHELL

9.1 INTRODUCTION

The reticulated shell exposed in the open air is subjected to various types of external loads and extreme events such as dead loads, corrosion, instability, earthquake, temperature change, fire and wind (Chinese code JGJ-61 2003). Various types of static and dynamic analyses have been conducted in the former chapters to evaluate the structural performance and assess the safety of the reticulated shell under dead loads, corrosion, instability and earthquake. The structural performance under temperature change, fire and wind shall be investigated in this chapter. The observations made in the analyses of corrosion damage reveal that the degradation of steel material in corrosive environment is important for the durability of the structure as a whole and may lead to structural damage accumulation. The strong external excitations such as earthquake may cause excessive vibration and lead to the potential damage or destruction of the reticulated shell. In addition, extreme events such as instability may lead to structural damage and even collapse. If the accumulated damage of the reticulated shell cannot be timely detected, the structural safety will be threatened and the damage may finally cause the partial or entire collapse of the shell, resulting in economic loss and fatal casualty. Therefore, it is considered prudent to install health monitoring system in the reticulated shell to monitor the environment, external loads and structural responses. In addition, control
system is also desirable to abate the excessive vibration of the shell under intensive external excitations.

The health monitoring system for a civil engineering structure is normally designed after the performance of the structure under various external excitations and extreme events is fully appraised. Then, the environmental factors and load information affecting the structural performance and the crucial parts of the structures with substantial responses are determined for the design of the type, number and distribution of various sensors. The information of environment, loads and structural responses collected from the installed sensory system is utilized to update the analytical models for assessing structural safety. Based on the observations made in the structural assessment, additional control system may be installed to provide additional safety provision if the structure cannot avoid the damages caused by intensive external excitations. Simultaneously, corresponding sensory system associated with the control system may also be installed to meet the operational requirements of the control process.

Health monitoring and vibration control of civil engineering structures under harsh environment have been actively investigated in recent years. In the aspect of structural health, monitoring systems consisting of sensors, data acquisition and data analysis have been developed and implemented in civil engineering structures to identify their dynamic characteristics and parameters and to detect their possible damage after extreme event or long-term service (Aktan et al., 2000; Wong et al., 2000). In the aspect of vibration control, several technologies have been developed for civil engineering structures to reduce their excessive vibration caused by strong
wind, severe earthquakes or other disturbances (Housner et al., 1997; Spencer et al., 2003). Semi-active control technology is now receiving considerable attention from engineering professionals because it offers the reliability of passive control systems and at the same time maintains the versatility and adaptability of active/hybrid control systems with much lower power requirement. Semi-active control technology is also studied in this thesis for mitigating the seismic excitation of the reticulated shell using semi-active friction dampers. To put semi-active control technology into effect, sensory system and data acquisition and transmission system are required in addition to semi-active control devices and control algorithm. Although both health monitoring system and vibration control system need sensors, data acquisition and transmission for real implementation, they have been treated separately in most of the investigations according to the primary objective pursued. This separate approach is not cost-effective if the reticulated shell does require both health monitoring system and vibration control system. In this regard, the conceptual design of an integrated health monitoring and vibration control system is carried out in this chapter by taking the reticulated steel shell as an example with the aim of updating analytical models, identifying structural parameters, assessing structural safety, guiding maintenance and repairing work, and activating control devices to protect the structure against extreme loading.

Various types of static and dynamic analyses should be carried out in order to understand the shell performance and assess its safety under external excitations and extreme events for developing integrated monitoring and control system. The static responses under dead loads and the modal characteristics of the shell are analyzed in Chapter 3. The damage evaluation of the reticulated shell subjected to corrosion
damage is also conducted in Chapter 4. The instability analysis is carried out in Chapter 5 to reveal the dangerous regions suffered from sudden stiffness reduction. The dynamic responses of the reticulated shell under earthquake are analyzed in Chapter 7 to explore the structural performance under intensive external excitation. The structural performance under ambient temperature change and fire is first examined in this chapter. Because the reticulated shell is light and flexible and it tends to deflect and oscillate under wind loads, the wind-induced responses analysis of the reticulated shell is also carried out in this chapter. After that, the structural behaviour, stability and safety of the reticulated steel shell under dead loads, wind loads, earthquake, temperature change, fire and corrosion are summarised. Based on these findings, various types of sensors are selected to measure climatic changes, atmospheric contaminants, material corrosion, wind, earthquake, structural responses and control forces among others. The numbers and locations of the sensors and control devices are also specified. Two databases are established to collect the information from the sensors and the inspection respectively. The main objectives of installing the integrated system are demonstrated based on the information collected and the layout of the integrated system is illustrated in detail in this chapter.

### 9.2 Shell Responses Due to Ambient Temperature Change

The reticulated shell exposed in the open air serves as a roof facility and works under large ambient temperature change during its long-term service period. This shell is an indeterminate structure with many redundant constraints. The substantial change in ambient temperature may lead to the stress redistribution and structural deformation (Chinese code JGJ-61 2003) because of redundant constraints. Therefore, it is necessary to examine the structural performance due to ambient temperature change.
The observations made in this analysis are relevant and useful in the design of the integrated monitoring and control system. Based on the structural theory (Bathe 1996), the structural responses induced by temperature change can be analyzed by using finite element method (FEM). Because the change in ambient temperature is actually quite slow, temperature change of structural members is normally regarded the same as that of ambient temperature and the temperature field of the structural members is assumed uniform. Following the principle of FEM, temperature change of the reticulate shell can be taken as a kind of external load in analyzing the structural performance. A reference temperature is normally selected as the baseline for the computation of the temperature change of shell members. The temperature-induced strains of shell members can be determined by multiplying the temperature change and the coefficient of thermal expansion of steel material. Then, the equivalent static loads (namely temperature loads) can be computed using the temperature-induced strains and the structural physical properties based on FEM. The deformation and stress distribution of the shell due to temperature change are computed by solving the static equation under the equivalent static loads, which is similar to the process adopted for structural static analysis under dead loads. The reticulated shell investigated in this thesis is located in Shijiazhuang City and the range of ambient temperature change in this city is approximately from -10 to 50 degrees Celsius in a year. This temperature variation range is adopted in the numerical computation. The reference temperature is taken as 20 degrees Celsius which is actually the normal atmospheric temperature. The analysis is carried out by using the commercial software ANSYS.
The variation of nodal displacement and maximum stress of members with temperature change under rigid constraints is first computed without considering the dead loads (See Figure 9.1). The curves in Figure 9.1 (a) demonstrate that the displacement of nodes on circle 5 is smaller than that of on other circles. Because the members on circle 6 are restricted by the boundary constraint, the stresses of these members are much larger than those of other circles. Considering the effects of boundary conditions on nodal displacement and member maximum stresses under temperature change, the structural responses under joint constraints are also computed and shown in Figure 9.2. The response comparison (nodal displacement and member maximum stresses) between rigid and joint boundary conditions demonstrates that their difference is quite small. Compared with results obtained only under dead loads, one can find that the nodal displacement caused by small temperature change (within 5 degrees Celsius) is close to that induced only by dead loads. The nodal displacement and member maximum stress under rigid constraints are also computed by considering both temperature change and the dead loads (See Figure 9.3). The results in Figure 9.3 demonstrate that the nodal displacement gradually increases with the increasing temperature change. If the temperature change is quite large, for instance 30 degrees Celsius, the nodal displacement is actually much larger than that only under dead loads. However, the stresses of most members induced by temperature change are quite small except those members on circle 6. This is because these members are fully restricted by the boundary constraints. Based on these understandings, several ambient and member temperature sensors shall be installed on the reticulated shell to monitor both ambient temperature and member temperature for the evaluation of the structural performance under
temperature change. The location of temperature sensors will be discussed later in the design of the integrated monitoring and control system.

9.3 SHELL PERFORMANCE IN A FIRE

Reticulated shells are commonly served as roofs for many public facilities such as exhibition centers, railway stations, etc. Owing to the human activity, the shell may carry a risk of potential fire hazard. Therefore, the understanding of shell performance under extreme events such as a fire is necessary for assessing the structural safety. The performance of the reticulated shell in a fire is examined in this section in order to obtain relevant and useful information for the design of the integrated system developed in this thesis. The procedure for analyzing the shell responses in a fire is basically illustrated and then the numerical results are provided and discussed.

9.3.1 Analytical procedure

The mechanical properties of steel material are very sensitive to high temperatures. In a fire, the Young’s modulus and yield strength of steel material will dramatically reduce with the increase in temperature. This loss of physical properties will inevitably lead to the reduction of load bearing capacity and possible structural collapse. Therefore, the fire analysis is essential for steel space structures for their safety assessment. Following structural theory (Bathe 1996; ANSYS 2003), the analysis of the reticulated shell in a fire can be briefly divided into four steps. Firstly, the ambient temperature field of the reticulated shell in a fire is determined by adopting well-developed prediction model. Secondly, the temperature distribution in
the structural members in a fire is computed based on the ambient temperature field and physical properties of steel material. The temperature distribution in the structural members is actually determined by solving the equation of heat conduction of the structural members based on FEM. Thirdly, the mechanical properties of steel material such as Young’s modulus and yield strength are updated based on the obtained temperature distribution of structural members and the finite element model of the reticulated shell are reconstructed. Finally, the structural responses under dead loads and temperature change of structural members are computed to evaluate the structural safety in the fire. The last step is similar to the process adopted for determining structural responses induced only by ambient temperature change.

The ambient temperature field of the reticulated shell in a fire can normally be estimated using the standard temperature-time curve proposed by International Standard Organization (ISO) following combustion tests (ISO 1985). This widely adopted temperature-time curve in fire analysis (ANSYS 2003) can be expressed as

\[ T = T_0 + 345 \log_{10} (8t + 1) \]  

(9.1)

where \( T \) is the ambient temperature (degree Celsius) in a fire; \( T_0 \) is the initial ambient temperature which is adopted as the normal atmospheric temperature (20 degrees Celsius) in this thesis; \( t \) is the time duration from the beginning of a fire (min).

In a fire, the released heat energy from combustion transfers to the surface of structural members through heat radiation, heat flux and heat conduction (Siegal and Howell 1981). Then, the heat energy transfers from member surface to its internal parts through solid heat conduction. The temperature distribution of a member is time-varying and can be numerically computed by solving the equation of heat
conduction based on ambient temperature field and physical properties of steel material following FEM (Bathe 1996, ANSYS 2003). After that, the important mechanical parameters of yield strength and Young’s modulus of steel material under different temperature fields are computed. The experimental researches across the world reveal that the variation of steel strength and Young’s modulus is not significant within 250 degrees Celsius. After that, obvious plastic deformation of steel material can be observed. In these circumstances, the magnitude of yield strength and Young’s modulus substantially reduces. Practically, many models of yield strength and Young’s modulus have been put forward in the past three decades such as ECCS model, Australia AS4100 model, Eurocode model, BS5950 model, etc. The model proposed by European Steel Association, ECCS model, is widely accepted and therefore adopted in the fire analysis carried out in this section (ECCS 1983). Following this model, the changes in yield strength and Young’s modulus of steel material with material temperature are expressed as

\[ \frac{f_{yT}}{f_y} = 1 + \frac{T_s}{767 \ln \left( \frac{T_s}{1750} \right)} \]  

(9.2)

\[ \frac{E_{yT}}{E} = -17.2 \times 10^{-12} T_s^4 + 11.8 \times 10^{-9} T_s^3 - 34.5 \times 10^{-7} T_s^2 + 15.9 \times 10^{-5} T_s + 1 \]  

(9.3)

where \( T_s \) is the temperature of steel material (degree Celsius); \( f_{yT} \) and \( f_y \) are the yield strengths of steel material at the temperature \( T_s \) and the normal atmospheric temperature respectively; \( E_{yT} \) and \( E \) are the Young’s modulus of steel material at the temperature \( T_s \) and the normal atmospheric temperature respectively. Based on the updated mechanical properties of structural members, the finite element model of the reticulated shell is reconstructed and the structural responses under both temperature
changes and dead loads are computed in order to evaluate the structural safety in a fire.

9.3.2 Numerical investigation

The responses of the reticulated shell in a fire are numerically investigated in this section by using the commercial software package ANSYS. Because a fire may occur at random, it is conservative to assume that the entire shell is subjected to a fire. The temperature model, ISO834, expressed in Equation (9.1) is adopted to simulate the ambient temperature field of the shell in a fire. The initial ambient temperature is selected as the normal atmospheric temperature (20 degrees Celsius). For simplicity, it is commonly accepted to assume that a single component such as a bar, beam or column is assumed to have uniform temperature field in its axial direction. This assumption makes it possible to simplify the member temperature field from three dimension (3D) to two dimension (2D) (ANSYS 2003). The 2D finite element model of the member section is constructed and plotted in Figure 9.4. The model for radial or circular member consists of 509 shell elements and the counterpart for skew member is 591. Because a fire is assumed to occur indoors, the lower parts of section will be directly affected by the heat radiation from the fire while the upper parts will not be affected by the heat radiation from the fire. The reticulated shell is assumed to be subjected to both the fire and the dead loads.

The member temperature fields at different time instants are computed and plotted in Figures 9.5 (a) to (c) for radial or circular members and Figures 9.5 (d)-(f) for skew members. Clearly, the temperature distributions of the two member sections are different to some extent. The temperatures of the lower parts are much larger than
those of the upper parts of member section. This is because the lower parts of section are directly affected by the heat radiation from the indoor fire and these parts absorb more heat energy than the upper parts. Owing to the small thickness of structural members, the temperature difference between the outer and inner surface of the member is very small. Similar observations can also be made from the temperature fields of skew member. The reduction of mechanical properties of structural members in the fire shall inevitably produce the force redistribution and nodal deformation of the entire shell. The nonlinear static analysis is carried out to examine the structural deformation under dead loads in the fire. For simplicity, the equivalent material parameters for a certain member are computed for response analysis

\[
E_{eq}(t_j) = \frac{\sum_{i=1}^{ns} E_i(t_j) \cdot A_i}{\sum_{i=1}^{ns} A_i}
\]

(9.4)

\[
f_{y,eq}(t_j) = \frac{\sum_{i=1}^{ns} f_{y,i}(t_j) \cdot A_i}{\sum_{i=1}^{ns} A_i}
\]

(9.5)

where \(E_i(t_j)\) and \(f_{y,i}(t_j)\) are the Young’s modulus and yield strength of the \(i\)th element at time instant \(t_j\) for a certain member; \(E_{eq}(t_j)\) and \(f_{y,eq}(t_j)\) are the equivalent Young’s modulus and yield strength of the entire member at time instant \(t_j\) for a certain member; \(A_i\) is the area of the \(i\)th element for a certain member; \(ns\) is the number of elements of a certain member.

The finite element model of the reticulated shell is reconstructed by using the updated yield strength and Young’s modulus for the examination of the structural responses. In the discussion about the analytical results, the peak temperature at a
The deformation of the reticulated shell under different peak temperatures of members is plotted in Figure 9.6. Before the peak temperature reaches about 700 degrees Celsius, the displacement is relatively small. If the peak temperature reaches about 700 degrees Celsius, the structural deformation rapidly increases and the shell is close to its collapse. At about 800 degrees Celsius, the buckling of a node between circles 2 and 3 occurs and the maximum vertical displacement of the shell at this temperature is about 1.114 m as shown in Figure 9.6 (d). Consequently, the destructive regions rapidly expand and the entire shell collapses eventually. To understand the performance of the shell subjected to both the fire and dead loads, one can find that with the rapid increase in member temperature, the load bearing capacity of the reticulated shell reduces significantly. This loss of load bearing capacity will cause the instability of some members and eventually lead to the collapse of the entire shell.

It is found that the reticulated shell will collapse at about 800 degrees Celsius. This temperature will be reached after about 3.5 hours from the beginning of the fire following the analytical results. Therefore, the time duration of the collapse process of the shell in the fire can be approximately estimated as 3.5 hours. It is necessary to install some fire alarm devices to monitor the possible fire before the rapid collapse of the entire shell. The fire alarm devices shall send alarm signals for the maintenance party to take urgent measures. The location of fire alarm devices will be discussed later in the design of the integrated monitoring and control system.
9.4 WIND-INDUCED RESPONSES OF RETICULATED SHELL

Reticulated shells are generally light and flexible and they tend to deflect and oscillate under turbulent wind loads. As the span increases, the natural frequencies generally decrease and the shells become more vulnerable to resonant excitation under wind excitation. Therefore, the numerical analysis is carried out in this section to evaluate the performance of the reticulated shell under wind loads.

9.4.1 Wind loads

Wind-induced dynamic responses of reticulated shells or domes have been studied by several researchers. For example, Mataki et al (1988) investigated the structural characteristics and wind resistant design of a low-rise cable-reinforced air-supported dome, based on a wind tunnel measurement of wind pressures as well as on a field measurement of wind-induced response of a large-scale model. Their results indicate that the response can be computed by applying a quasi-static approach to the estimation of the wind loads. Ogawa et al (1989) made a statistical analysis on the dynamic responses of an air-supported spherical shell, using the above-mentioned model of the pressure field. Current research work demonstrates that the dynamic responses of reticulated shells can be accurately computed by using the time history of wind pressures at many points (Uematsu et al 2002b). However, this procedure is somewhat complicated and time consuming because the analytical procedure should be carried out based on the data of wind pressure obtained from wind tunnel test. In this regard, Uematsu et al (2002b) investigated the design wind loads for the structural frames of the shells with long spans. The wind tunnel experiment was carried out to obtain wind pressure data to be used for the dynamic response analysis of a full-scale shell in the time domain. The characteristics of the dynamic responses
were investigated in detail. It is found that the responses are almost quasi-static and
the resonant effects on the dynamic responses are relatively small. Then, a series of
quasi-static analyses, in which the inertia and damping terms are neglected, were
carried out for a wide range of the shell’s geometry. Based on the results, they
discussed the equivalent pressure coefficients that reproduce the maximum load
effects. The focus is on the applicability of the gust effect factor approach. Finally,
they provided an empirical formula for the gust effect factor and a simple model of
the pressure coefficient distribution. The validity of the formula and analytical model
was confirmed and they concluded that the design wind load can be estimated by the
gust effect factor approach.

For the reticulated shell examined in this thesis, the wind tunnel test is not carried out
in this study to obtain the pressure data for dynamic analysis due to the limitation of
research funds and time. Because the wind-induced responses are almost quasi-static
and the resonant effects on the dynamic responses are relatively small for reticulated
shells, the gust effect factor approach developed by Uematsu et al (2002b) is utilized
in this section. This procedure provides a convenient approach to understand the
shell performance under wind excitations.

Figure 9.7 displays the side view of the reticulated shell investigated in this study. $D$
is the span of the shell. $f/D$ is the rise/span ratio. $h/D$ is the eaves-height/span ratio.
Based on the observations made from wind tunnel tests, Uematsu et al (2002b)
reported that the variation of gust effect factor $G_f$ with the turbulence intensity $I_{u,H}$
of the approaching flow at the mean shell height $H$ is similar to that used for circular
flat roofs. Uematsu et al (2002a) provided an empirical formula for the gust effect factor of circular flat roofs $G_{f,\text{flat}}$ as follows

$$G_{f,\text{flat}} = 1 + g \cdot r_f \cdot R$$  \hspace{1cm} (9.6)

$$g \approx \sqrt{2 \ln 600 f_1} + \frac{0.577}{\sqrt{2 \ln 600 f_1}}$$  \hspace{1cm} (9.7)

$$r_f = 3.41 I_u^2 \cdot \exp \left( 0.04 \frac{D}{H} \right) + 0.12$$  \hspace{1cm} (9.8)

where $g$ is the peak factor; $r_f$ is the ratio of the RMS to the mean modal force coefficient of the first axisymmetric mode; $R$ is the resonant magnification factor; and $f_1$ is the natural frequency of the first axisymmetric mode. When $D/H > 7$, Uematsu et al (2002a) suggested that one may substitute 7 for $D/H$ in Equation (9.8). $R$ is regarded as 1.0 when dealing with the quasi-static responses. Uematsu et al (2002b) suggested that the estimation of the gust effect factor $G_j$ for the reticulated shell can be performed by using the formula for $G_{f,\text{flat}}$ with some modification

$$\frac{G_j}{G_{f,\text{flat}}} = \begin{cases} 4(C_{\text{max}} - 1) \frac{h}{D} + 1.0 & \text{for} \ 0 \leq \frac{h}{D} \leq 0.25 \\ C_{\text{max}} & \text{for} \ 0.25 < \frac{h}{D} \leq 1.0 \end{cases}$$  \hspace{1cm} (9.9)

$$C_{\text{max}} = \begin{cases} 1.0 + 4 \frac{f}{D} & \text{for} \ 0 \leq \frac{h}{D} \leq 0.1 \\ 1.8 - 4 \frac{f}{D} & \text{for} \ 0.1 < \frac{h}{D} \leq 0.2 \end{cases}$$  \hspace{1cm} (9.10)

Using Equations (9.6) to (9.10), the gust effect factor $G_j$ for the reticulated shell with $f/D = 0-0.2$ and $h/D = 0-1.0$ can be easily estimated. The equivalent pressure coefficients reproducing the same load effects that the practical pressure distribution induces are also provided by Uematsu et al (2002b). Figure 9.8 shows the model of the pressure coefficient distribution on the reticulated shell provided by Uematsu et
The shell is divided into four regions, labeled as ‘R1’ to ‘R4’, and the pressure coefficient in each region is assumed constant. The value of the equivalent pressure coefficient in each region is given by spatially averaging the pressure coefficient distribution over the region. The results of the equivalent pressure coefficients $C_{p1}$ to $C_{p4}$ for the four regions demonstrate that the values were not affected by the turbulence intensity of the approaching flow significantly. They depend on $f/D$ as well as on $h/D$. The values of $C_{pi}$ ($i=1-4$) for $h/D=0, 0.25$ and $1.0$ are provide by Uematsu et al (2002b) and listed in Table 9.1. Therefore, the design wind loads for each region of the reticulated shell can be expressed as

$$w_{p,i} = w_H G_j C_{pi} \quad (i = 1, 2, 3, 4)$$

(9.11)

where $w_H$ is the wind pressure at the mean shell height $H$.

### 9.4.2 Wind-induced responses

Based on the quasi-static approach proposed by Uematsu et al (2002b), the wind-induced responses of the reticulated shell are analyzed in this section. An approximate formula to estimate the magnitude of the turbulence intensity at the mean shell height $H$ is

$$I_{uH} = 0.1 \cdot \left(\frac{\delta}{H}\right)^\alpha$$

(9.12)

where $\delta$ is the gradient height; $\alpha$ is the power exponent. For the reticulated shell examined in this study, the $\delta = 400m$ and $\alpha = 0.26$. The mean shell height $H$ is $13.126m$. The turbulence intensity $I_{uH}$ of the approaching flow at the mean shell height $H$ is computed as $0.2431$. The basic design wind pressure in Shijiazhuang City is $0.3kN/m^2$ ($10m$ height). By using the power law for the wind velocity (power
exponent is adopted as 0.28), the wind pressure at the mean shell height $H_{w_H}$ is computed as $0.349kN/m^2$. The shell span $D$, eaves-height $h$, and rise $f$ are 64.866m, 9m, and 9.916m respectively. The first natural frequency of the first axisymmetric mode of the reticulated shell is 4.066 Hz. Based on the above mentioned analytical process, the gust effect factor $G_f$ for the reticulated shell is computed as 2.7555. In addition, the equivalent pressure coefficients $C_{p_1}$ to $C_{p_4}$ can be obtained by interpolating the data in Table 9.1. Therefore, the design wind loads for the reticulated shell can be determined.

The quasi-static responses of the reticulated shell are computed based on the determined wind loads. In the analysis, the approaching flow is assumed in the global $x$ direction. The absolute nodal displacement and member maximum stresses are computed and plotted in Figures 9.9 and 9.10 respectively. The curves in Figure 9.9 demonstrate that vertical displacement of nodes is much larger than the horizontal displacement. The maximum displacement of the reticulated shell is only $1.05\,\text{cm}$. Figure 9.10 demonstrates that the maximum member stress of the reticulated shell under wind loads is about $33\,\text{MPa}$ which is much smaller than the yield strength of the member material ($235\,\text{MPa}$). The analytical results also reveal that the axial responses of structural members are substantially larger than bending responses. These observations are the same as those made in static analysis under dead loads and dynamic analysis under earthquake. As reported by Uematsu et al (2002b), the wind-induced responses of the reticulated shell are almost quasi-static and the resonant effects on the dynamic responses are small. The structural responses under equivalent static wind loads are quite close to those obtained from time history analysis using the measured data. Therefore, the obtained nodal displacement and
member stresses of the reticulated shell under equivalent static wind loads is close to
the actually maximum values obtained using dynamic wind pressure information.
The results of the reticulated shell from the quasi-static analysis demonstrate that
wind-induced responses of the reticulated shell are smaller than those under El
Centro earthquake normalized to 0.22g. It is also observed that the member stress
and structural displacement under wind load are really small. Therefore, it is
unnecessary to conduct vibration control for the reticulated shell under wind loads.

9.5 Design of Integrated Monitoring and Control System

9.5.1 Objectives

As discussed before, various static and dynamic analyses have been conducted in the
former chapters and sections for assessing the safety of the reticulated shell under
external excitations and extreme events. The static responses under dead loads and
the modal characteristics of the shell are computed in Chapter 3 respectively. The
damage evaluation of the reticulated shell subjected to coupled atmospheric
corrosion and SCC is conducted in Chapter 4. The instability analysis is also carried
out in Chapter 5 to reveal the possible regions suffered from sudden stiffness
reduction. The dynamic responses of the reticulated shell under earthquake are
analyzed in Chapter 7 to explore the structural performance under intensive external
excitation. In addition, the shell performance under variation of ambient temperature,
fire and wind loads are computed in this chapter in order to understand the effects of
environment and extreme events on the reticulated shell. The analytical observations
indicate that the shell may have some damage in harsh environment. In order to
assure the structural safety during its long-term service period, the health monitoring
system should be installed in the reticulated shell to monitor the environment, external excitations and structural responses. In addition, effective approaches should be proposed to identify the structural damages and mitigate the structural responses. In this regard, two damage detection approaches are proposed in Chapter 5 in order to detect sudden damage of a structure based on a new index in the time domain. The first approach can be implemented in an online health monitoring system without any time delay. The second approach using empirical mode decomposition (EMD) gains an additional insight into the feature of sudden damage and can also provide other information of damage in the time-frequency domain in conjunction with the Hilbert transform. An approach based on the additional stiffness is proposed in Chapter 7 using natural frequencies and mode shapes for the detection of slow damage events. In addition, the damage detection of the reticulated shell with semi-active friction dampers is also carried out in the time domain by developing a sensitivity-based detection approach based on the measured control forces in Chapter 8. The vibration control of the reticulated shell subjected to earthquake is conducted by using the semi-active friction dampers in Chapter 7. Based on the proposed approaches for damage detection and response control of the reticulated shell, reasonable health monitoring system and vibration control system can be designed and installed in the shell respectively.

Vibration control and health monitoring of civil engineering structures have been treated separately in most of the investigations. This separate approach is not cost-effective if structures do require both vibration control system and health monitoring system. Therefore, an integrated health monitoring and vibration control system is developed for the reticulated shell to assess and structural safety and reduce vibration.
in harsh environment. This integrated system is conceived as a complicated system making use of the techniques of structural analysis, system identification and vibration control. It is intended that when completely developed it will be an integrated system used for the following purposes:

(1) To measure climatic changes, atmospheric contaminants, material corrosion, wind, earthquake and structural responses by selecting various and proper types of sensors which forms a sensory system, and to update various analytical models based on the information collected from the sensory system.

(2) To examine the structural behaviour of the reticulated shell under dead loads, wind, earthquake load, temperature change, fire and corrosion using the updated analytical models, to assess the structural functionality and safety, and to detect possible damage.

(3) To determine the crucial parts of the reticulated shell based on the observations made in structural analyses and safety assessment so that the planning of shell inspection work will be facilitated and the maintenance and repairing work will be guided. The collected information from field inspection work, in turn, will be utilized for refining the analytical models and assessing the structural safety.

(4) To activate the control devices for protecting the reticulated shell against excessive vibration if the structure is subjected to strong excitations. This process shall be carried out after effectively understanding the structural performance and features of external loads. Practical yet effective control strategy and sensory system should be adopted to facilitate the realization of the control process. In addition, effective algorithm for parameter identification and damage detection using these control devices shall be developed. The sensory system and control devices shall be
shared for both health monitoring and vibration control to develop a practical and economical system.

(5) To advance the knowledge and application of sensory technology, corrosion science, identification techniques, control theories, structural analyses and damage detection for the operation and maintenance of civil engineering structures in the future.

9.5.2 System design

Based on the above mentioned objectives, the integrated monitoring and control system should be designed for achieving the goals of structural monitoring and response control. The sensory system should be firstly determined to collect various environmental information, load information, and structural responses. In addition, necessary field inspection work should be carried out to provide more useful information of the shell. The collected information of environment, loads and structural responses should be stored in different databases and further analyzed based on different models for safety assessment. Corresponding system operation shall be carried out to activate the control devices for protecting the structure against the extreme loads. Therefore, all these aspects in the design of the integrated health monitoring and vibration control system will be conducted and illustrated in this section.

9.5.2.1 Sensory and control systems

As discussed above, several damage detection and vibration control approaches have been proposed in the former chapters for the reticulated shell. To effectively conduct identification and control approaches, necessary information of environment, loads
and structural responses should be collected. The control devices shall be installed to
abate the structural vibration under external excitations and carry out the system
identification. Sometimes, if the information collected from the installed sensory
system cannot fully meet the requirements of system identification and safety
assessment, portable sensory system shall be temporarily adopted for information
collection. Effective sensory and control system should be designed and constructed
to provide relevant and useful information for the monitoring and control processes.
In this regard, the sensory and control system of the reticulated shell comprises: (1)
sensors for monitoring environment and loads; (2) sensors for monitoring structural
responses; (3) control devices and associated sensors; and (4) portable sensory
system for supplementary information. These different types of sensors and control
devices will be discussed in the following.

(1) Sensors for monitoring environment and loads
Several sensors shall be installed on the shell to measure climatic changes,
atmospheric contaminants, material corrosion, wind, earthquake and fire. The
structural responses under dead loads are computed in Chapter 3. The damage
evaluation of the reticulated shell under corrosive environment is also actively
conducted. The evaluation of atmospheric corrosion damage shall be carried out by
monitoring the atmospheric environment and material corrosion. The bearings are the
major supporting components of the shell which have relatively complicated
configuration and can easily accumulate rain and moisture than other parts of the
reticulated shell. This moist environment easily leads to the corrosion damages. The
observations made in the sensitivity analysis in Chapter 3 reveal that the members
close to the shell centre are the most sensitive components to the section reduction
caused by atmospheric corrosion. These members are therefore the crucial parts in the corrosion monitoring. In these regards, four corrosion sensors are placed on shell centre and bearings respectively as shown in Figure 9.11. Because the material corrosion will be substantially affected by the environmental moisture, it is thus essential to place hygrometers with the same number and distribution as those of corrosion sensors for monitoring the humidity of structural components. An ombrometer and an acidimeter are required to monitor the precipitation, precipitation days and rain PH value. Furthermore, a pyranometer is installed to record the changes in sunshine hours. Because the size of the reticulated shell is not very large, only a group of contaminant sensors (SO$_2$ sensor, NO$_2$ sensor and Cl$^{-}$ sensor) is installed for the monitoring of the atmospheric contaminants. Considering the contents of the atmospheric contaminants directly relating to the velocity and direction of air flows and wind loading which will be discussed later, an anemometer is installed at the top of the shell to record the in-situ wind information.

The reticulated shell exposed in the open air is served as the roof facilities and works under ambient temperature change. Therefore, the structural performance under variation of ambient temperature (Chinese code JGJ-61 2003) is examined in this chapter. Based on the obtained understandings, it is found that the temperature change may cause stress redistribution and deformation of the shell. Therefore several temperature sensors shall be installed on the reticulated shell to monitor both the ambient and member temperatures for the evaluation of the structural performance due to temperature change. In addition, the material corrosion states will be affected by temperature. It is thus proposed to place one ambient temperature sensor and ten member temperature sensors on the shell (see Figure 9.12) in order to
monitor the changes in ambient and member temperatures. The obtained information is utilized in the analyses of temperature response and corrosion damage of the reticulated shell.

The responses of the reticulated shell in a fire are also investigated in order to understand the structural performance under extreme fire events. The results demonstrate that the time duration of the structural collapse (about 800 degrees Celsius) in the fire is about 3.5 hours. Therefore, it is necessary to install some sensors to monitor the potential fire before the rapid collapse of the entire shell and send alarm signal for the maintenance party to take urgent measures. Eight fire alarm devices are installed on the shell as shown in Figure 9.13 to detect the fire and send immediately alarms. Because the occurrence of fire is actually at random, the fire alarm devices are evenly distributed in the spatial range of the reticulated shell.

The reticulated shell is generally light and flexible and it tends to deflect and oscillate under turbulent wind loadings. Therefore, the response analysis is carried out to evaluate the structural performance under wind excitations. The results from quasi-static analysis demonstrate that the dynamic responses of the reticulated shell under wind excitations are small. It is unnecessary to conduct vibration control for the concerned shell under wind excitations. After a long-term service, the reticulated shell may have wind-induced fatigue damages. Therefore, several wind pressure sensors are distributed on the shell to measure the wind pressures (see Figure 9.14) for establishing exact wind load assessment model. Following the wind pressure distribution shown in Figure 9.8 and considering the unknown direction of wind flow, the wind pressure sensors are placed in two orthogonal directions. In each direction,
five sensors are installed to measure the wind pressures in four typical regions as described in Figure 9.8. In addition, a seismometer is placed on the ground to collect the earthquake information in all the three global directions as shown in Figure 9.14.

(2) Sensors for monitoring structural responses

The sensors for monitoring responses of the reticulated shell include accelerometers, displacement transducers, laser displacement transducers and strain gauges. The instability of the structural members may cause the sudden stiffness reduction of the reticulated shell. The sudden reduction of structural stiffness will cause a discontinuity in acceleration response time histories recorded in the vicinity of damage location at damage time instant. In this regard, several damage detection approaches are proposed in this study to detect sudden damage events of the reticulated shell in the time domain. The numerical results from Chapter 5 indicate that the members close to the shell centre firstly suffer from the sudden damage events compared to other members. In addition, the static and dynamic analyses of the reticulated shell demonstrate that the axial responses of members are much larger than bending responses. Thus, the axial stiffness of the member is important to the structural load bearing capacity. Therefore, the sudden stiffness reduction of the reticulated shell refers to the reduction of member axial stiffness. As analyzed in Chapter 5, for the sudden reduction of member axial stiffness, the damage information fully exists in the acceleration responses in member’s axial direction. Only one accelerometer is required for a member to monitor the possible sudden stiffness reduction. Therefore, the members close to shell vertex are installed with accelerometers in member’s axial direction to record the acceleration response in
local coordinate system for detecting the possible sudden damage events as shown in Figure 9.15.

The results from static and dynamic analyses carried out in this thesis demonstrate that the vertical displacement of the reticulated shell is much larger than horizontal displacement. Therefore, the vertical motion of the shell is monitored by the laser displacement transducers. The laser displacement transducers are mounted at five locations as displayed in Figure 9.16. As discussed above, the bearings are the major supporting components of the shell which have relatively complicated configuration and can easily accumulate moisture and suffered from corrosion damage in the open air. Therefore, three displacement transducers are needed for examining the potential deformation of the bearings due to the possible bearing damage (See Figure 9.16).

As mentioned above, a reasonable way for monitoring and assessing the potential damage due to atmospheric corrosion is to install various sensors for monitoring the climatic conditions and atmospheric contaminants. In reality, strain gauges are also required to measure the strains of important structural members and further determine their stresses. As mentioned in Chapter 3, the member stresses will be affected by the section loss due to atmospheric corrosion. In addition, the results from static analysis and seismic analysis carried out in Chapters 3 and 7 demonstrate that the radial members and circular members in the first three circles are the major load bearing components. Therefore, the stresses of these members should be effectively measured. The strain gauges can be placed with relatively larger amount due to its low cost. In this regard, sixty members in the first three circles are installed with strain gauges on both member body and connection joints as shown in Figure
Furthermore, to check the stress changes induced by potential bearing damage, eight radial members close to the bearings are also installed with strain gauges for strength examination (See Figure 9.17).

(3) Control devices and associated sensors

Following the investigation on seismic mitigation and damage detection carried out in Chapters 7 and 8 utilizing semi-active friction dampers, the damper installation scheme No.2 as displayed in Figure 9.18 is adopted for the integrated monitoring and control system. As discussed in Chapter 7, the information of damper slippage and control force should be measured to conduct vibration control based on local feedback control strategy. In addition, the control force of the semi-active friction damper should be measured to carry out the parameter identification and damage detection of the controlled shell using the presented time domain approach in Chapter 8. Therefore, a force transducer is installed in series with each friction damper to measure the control forces. Moreover, a displacement transducer is also incorporated into each semi-active friction damper to record the information of damage slippage.

(4) Portable sensory system

If the information collected from the installed sensory system cannot fully meet the requirements of system identification and safety assessment, portable sensory system shall be temporarily adopted for information collection in field measurement work. The field measurement work will consist of several types of measurements such as dynamic properties and corrosion media. For the damage detection, safety assessment and vibration control of the reticulated shell, the structural model
established utilizing initial design parameters should be updated based on the measured structural information. The well-accepted model updating approaches at present for civil engineering structures are carried out by mainly using measured natural frequencies and mode shapes. While the accelerometers installed in the shell are not able to provide enough information for the determination of mode shapes, some portable accelerometers shall be temporarily deployed to provide supplementary information. In addition, to obtain accurate modal information, controllable vibration exciters such as hydraulic vibrators can be temporarily employed to excite the structure. The obtained acceleration responses under controllable exciting source shall be utilized to identify structural dynamic characteristics for model updating, parameter identification and damage detection. As discussed in Chapter 4, the corrosion states of structural components shall be measured using corrosion sensors to assess the structural safety in corrosive environment after a long-term service. In addition, the chemical constitutions of corrosion media formed from the dissolution of air contaminants in accumulated rain and moisture shall be regularly tested through field measurement and simulated in laboratory to determine the threshold stress intensity level $K_{ISC\text{C}}$. The evaluation procedure of SCC shall be modulated based on the experimental value of $K_{ISC\text{C}}$. Therefore, some portable sensors shall be adopted in field sampling process. All the information obtained from the on-structure and portable sensory system should be effectively and rapidly collected through data transmission system to the work stations for storage and analysis.
9.5.2.2 Databases

The integrated monitoring and control system aims to monitor the environment changes and structural responses, control excessive vibration, and evaluate the safety of the reticulated shell in harsh environment. Various types of sensors are selected to measure the climatic changes, atmospheric contaminants, material corrosion, wind, earthquake, structural responses and control forces among others. The integrated system includes many kinds of sensors: acidimeter, anemometer, hygrometer, pyranometer, ombrometer, SO\textsubscript{2} sensor, NO\textsubscript{2} sensor, Cl\textsuperscript{-} sensor, corrosion sensors, temperature sensors, wind pressure sensors, fire alarm devices, displacement transducers, laser displacement transducers, accelerometers, strain gauges and force transducers. Before the structural analysis and safety assessment, the collected information from sensory system shall be stored in database for different usages. Therefore, all the field measurement data collected from both the permanent and portable sensory system are stored in the database for the measurement information.

As discussed in Chapter 4, periodical field inspection work is required to examine crack configuration of the reticulated shell whose results will be utilized to update the analytical model for coupled atmospheric corrosion and SCC. The observations made in primary analyses are relevant and useful for instructing the shell inspection work. The obtained inspection information, in turn, shall be utilized for refining analytical models and safety assessment procedure. In addition, the chemical constitution of this corrosive media can be regularly measured and simulated in laboratory to determine the threshold stress intensity level $K_{\text{ISC}}$. Therefore, another database is also designed to store the information collected from field inspection work.
Practically, various types of sensors are utilized to measure the climatic changes, atmospheric contaminants, material corrosion, wind, earthquake, structural responses and control forces. Because the sensory system is divided into four types, four sub-databases are designed to store the information in the measurement database in line with the different objectives of the information: (1) sub-database for environmental and load information; (2) sub-database for structural responses; (3) sub-database for information of control devices; and (4) sub-database for information from portable sensory system. The envisaged tasks in the formation and processing of these four sub-databases are described as follows:

(1) Sub-database for environmental and load information stores the measured information of climatic changes, atmospheric contaminants, material corrosion, wind and earthquake.

(2) Sub-database for structural responses stores the measured information of acceleration responses, deformation and strains of structural components.

(3) Sub-database for information of control devices stores the measured information of control forces and slippage of the semi-active friction damper during the vibration control and system identification process.

(4) Sub-database for information from portable sensory system stores the obtained information of field measurement work such as natural frequencies, mode shapes, corrosive media, etc.

9.5.2.3 Analytical models

To conduct the damage detection, safety assessment and the vibration control, various analytical models of environmental change, structural responses and external loads should be updated to accurately predict the structural performance. Based on
the collected information in the databases from on-structure measurement and field inspection work, several analytical models of the reticulated shell shall be updated for further analyses which include original finite elements model, prediction model for atmospheric corrosion, evaluation model for coupled atmospheric corrosion and SCC, the wind load model and the temperature distribution model. Then, the updated analytical models are utilized to carry out the information expansion process in order to obtain the responses of those members without the installation of sensors through theoretical analyses.

The natural frequencies and mode shapes of the reticulated shell can be identified based on acceleration responses obtained from portable sensory system through field measurement work. Then, the vibration-based model updating approaches can be applied to update the primary finite element model established using the original design parameters. The effectiveness of the updated finite element model should be investigated by examining the differences of analytical and measured results. The corrosion depth of some components can be measured through the installed corrosion sensors. In addition, the atmospheric contaminants and climatic information such as wind velocity and direction, ambient temperature, humidity, sunshine, precipitation and rain PH value can also be measured through various sensors. Therefore, the prediction model for atmospheric corrosion of steel components can be updated to meet the requirements of the reticulated shell in particular. As mentioned in Chapter 4, atmospheric contaminants may be dissolved in accumulated moisture and form the corrosive environment which may cause the SCC of structural components. Therefore, the chemical constitution of this corrosive media can be regularly measured and simulated in laboratory to determine the threshold stress intensity
level $K_{ISC}$. In addition, the crack configuration of structural components can be determined through field inspection work. Based on the refining information of the threshold stress intensity level $K_{ISC}$ and the crack configuration, the evaluation procedure of SCC of the reticulated shell can be modulated. Furthermore, if the contents of atmospheric contaminants vary substantially during short time periods, it is reasonably to increase the frequency of regular measurement on chemical constitution of corrosion media. Corresponding laboratory test is required to modify the value of $K_{ISC}$ and evaluation procedure of SCC. The data collected from the wind pressure sensors and anemometers concerns the characteristics of the wind on site which shall be applied to meet future analytical demands. The patterns of wind load distributions adopted in the theoretical analysis shall be updated based on the measured data. It will be the aim to provide practical yet accurate wind load data for calculating the wind-induced responses of the reticulated shell. Similarly, the changes of ambient and member temperatures of the shell can be measured to update the models of temperature loads for thermal analysis.

Utilizing the sensory system designed for the reticulated shell, only the responses of limited structural components can be measured due to the number limitation of sensors. Based on various types of updated structural and load models, the responses of members without the installation of sensors can be predicted through the information expansion process. The updated finite element model of the reticulated shell can be utilized to compute the structural displacement and member stresses under various excitations. For the atmospheric corrosion, the section loss of members can be estimated by using the updated prediction model of atmospheric corrosion. The stresses of all the members due to section loss caused by atmospheric corrosion
can be determined. The computational formulae for stress intensity factor should be
modulated based on the crack configuration measured from the field inspection.
Therefore, the SCC evaluation can be carried out for all the structural components.

9.5.2.4 System operation and layout

After updating the analytical models based on information collected from the sensory
system and field inspection, structural analyses shall be carried out to examine the
performance of the reticulated shell under various types of external loads and
extreme events. The crucial parts of the shell suffered from potential damages can be
determined based on the analytical results. Then, the system identification including
parameter identification and damage detection is carried out to explore the potential
structural damages. The structural safety therefore is assessed based on the
understandings from structural analysis and system identification. If the reticulated
shell is assessed to be lack of enough strength against intensive loads, the integrated
system shall send instructions to activate the control devices for vibration mitigation.
Therefore, the operation of the integrated system consists of four major steps:
structural analysis, system identification, safety assessment, and vibration control.

Based on the updated analytical models, various types of analyses shall be carried
out to understand the structural performance. The dynamic characteristics (natural
frequencies and mode shapes) of the reticulated shell can be computed by using the
updated finite element model. Static analysis is carried out to determine structural
deformation and stress distribution. The section loss and stress redistribution of
structural members due to atmospheric corrosion are determined by using the
updated prediction model. The structural natural frequency sensitivity to structural
member thickness is computed to assess the sensitivity of natural frequency to changes in member thickness due to atmospheric corrosion. The SCC of the reticulated shell is evaluated using the updated information of crack and threshold stress intensity factor. The stability analysis is conducted to explore the possible regions suffered from sudden damage events. The responses of the reticulated shell under wind and earthquake are also analyzed to evaluate the structural performance under external loads and extreme events. The temperature response analysis is conducted to evaluate the structural characteristics due to changes in environmental temperature and estimate the temperature distribution of the shell or predict any structural effects caused by temperature changes. The fire analysis is carried out to evaluate the load bearing capacity of the shell in a fire and to predict the structural failure states. Based on the observations made in various analyses, the crucial parts of the reticulated shell may suffer from potential damages will be determined. Then, the parameter identification and damage detection are conducted to examine the potential structural damages. Both sudden damage events and slow damage events shall be detected for the shell. Based on the understandings from structural analyses and system identification, the safety assessment is carried out to evaluate the structural functionality and serviceability. Finally, the control devices shall be activated to protect the reticulated shell against extreme loads if the structure is diagnosed to be lack of strength provisions.

Following the above discussion, the conceptual system layout of the integrated health monitoring and vibration control system for the reticulated shell is displayed in Figure 9.19. The flow chart demonstrates that the integrated system is developed based on structural theories, control techniques, identification algorithm, corrosion
knowledge and measurement technologies. For the integrated health monitoring and vibration control system, the data obtained from the sensory system and field inspection work should be effectively processed for system identification, safety assessment and vibration control. Hence, an effective software system shall be developed for data processing and archiving to manipulate the operation of the integrated system. Based on the obtained environmental, load and response information, various evaluation approaches shall be applied to assess the structural safety, abate structural responses and provide relevant and useful suggestions for the maintenance of the reticulated shell. The integrated health monitoring and vibration control system is developed for the reticulated shell through the application of corrosion science, identification techniques, control theories, structural analytical techniques and sensory technologies which may be beneficial to the structural operation and maintenance and be used in the design and construction of other steel space structures in the future.

9.6 SUMMARY

The conceptual design of integrated health monitoring and vibration control system for the reticulated shell is conducted in this chapter. The structural responses due to changes in ambient temperature and fire are firstly examined. The results reveal that the nodal displacement caused by small temperature changes (within 5 degrees Celsius) is close to that induced by dead loads. In a fire, the rapid increase in material temperature will significantly cause the loss of structural load bearing capacity and local instability under dead loads which will eventually leads to the collapse of the whole shell. The wind-induced responses are also computed to assess the structural safety. Compared with earthquake-induced responses, the responses under wind
excitations are relative small and it is unnecessary to conduct wind-induced vibration control for the reticulated shell. The observations of these analyses are combined with those from other static and dynamic analyses carried out in the former chapters to provide guidance for the conceptual design of integrated monitoring and control system.

The objectives of establishing the integrated system for the reticulated shell is illustrated. Various types of sensors are selected to measure climatic changes, atmospheric contamination, material corrosion, wind, earthquake, structural responses, and control forces among others. The numbers and locations of the sensors and control devices are also specified. The sensory system includes many kinds of sensors: acidimeter, anemometer, hygrometer, pyranometer, ombrometer, temperature sensors, SO₂ sensor, NO₂ sensor, Cl⁻ sensor, corrosion sensor, wind pressure sensor, seismometer, fire alarm devices, accelerometers, displacement transducer, laser displacement transducer, strain gauge and force transducer. The field inspection work required for structural safety assessment is illustrated. Two databases are established to archive the information from the sensors and the inspection respectively. Based on the collected information, the analytical models are required to be updated for structural analysis, system identification and safety assessment. Finally, the system operation and layout is illustrated in detail.
Table 9.1 Equivalent pressure coefficient $C_{pi}$ ($i=1-4$) in each region for different $h/D$

<table>
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<th>$h/D=0.25$</th>
<th>$h/D=1.0$</th>
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<td>R2</td>
<td>R3</td>
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<td>0.3</td>
<td>0.0</td>
<td>-0.2</td>
</tr>
<tr>
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<td>0.4</td>
<td>0.0</td>
<td>-0.4</td>
</tr>
<tr>
<td>0.20</td>
<td>0.5</td>
<td>0.0</td>
<td>-0.6</td>
</tr>
</tbody>
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Figure 9.1 Variation of nodal displacement and member maximum stress with temperature (rigid constraint).

Figure 9.2 Variation of nodal displacement and member maximum stress with temperature (joint constraint).
Figure 9.3 Variation of nodal displacement and member maximum stress with temperature under dead loads (rigid constraint).

Figure 9.4 2D finite element models of the member section:
(a) Radial or circular members; (b) Skew member.
Figure 9.5 Member’s temperature field at different time instants.
Figure 9.6 Structural deformation at different peak temperatures.

Figure 9.7 Side view of the reticulated shell.

Figure 9.8 Model of pressure coefficient distribution on the reticulated shell (Uematsu et al., 2002).
Figure 9.9 Absolute nodal displacement under wind loads.
Figure 9.10 Absolute member stresses under wind loads.

Figure 9.11 Distribution of sensors for corrosion monitoring.
Figure 9.12 Distribution of temperature sensors.

Figure 9.13 Distribution of fire alarm devices.
Figure 9.14 Distribution of wind pressure sensors and seismometer.

Figure 9.15 Distribution of accelerometers.
Figure 9.16 Distribution of displacement transducers and laser displacement transducers.

Figure 9.17 Distribution of strain gauges.
Figure 9.18 Distribution of semi-active friction dampers.
Figure 9.19 Layout of integrated health monitoring and vibration control system.
CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

10.1 CONCLUSIONS

This thesis pursues the understanding of the structural behaviour of steel space structures under various types of external loads including atmospheric and stress corrosion, the development of an innovative yet practical algorithm for structural damage detection, the combination of an health monitoring with vibration control towards a smart steel space structure, and the formation of integrated structural health monitoring and vibration control system for the best protection of steel space structures. The structural behaviour, stability and safety of the reticulated steel shell under dead load, wind load, earthquake load, temperature, fire and corrosion are investigated or summarised. The integrated health monitoring and vibration control system is developed in this thesis with the aim of updating analytical models, identifying structural parameters, assessing structural safety, guiding maintenance and repairing work, and activating control devices to protect the structure against extreme loading. A framework for the evaluation of potential damage due to atmospheric corrosion to steel space structures is proposed by combining corrosion knowledge, sensitivity analysis and nonlinear static computation. The research work on atmospheric corrosion damage is extended by involving SCC to provide an integrated procedure of evaluation and monitoring techniques on corrosion damages of steel space structures. The sudden damage caused by SCC or structural instability is investigated to effectively detect the time instant, location and severity of damage event. The features of signal discontinuities in the acceleration response time
histories recorded in the vicinity of damage location due to a sudden damage event are examined to propose an instantaneous damage index for sudden damage detection. In most of previous investigations, structural health monitoring and structural vibration control have been treated separately. This study presents an integrated procedure for health monitoring and vibration control of structures using semi-active friction dampers towards a smart structure. The concept of integrated health monitoring and vibration control systems using semi-active friction dampers is introduced by means of a shear building subjected to earthquake excitation. It is then applied to the reticulated steel shell with some adjustments in the control algorithm and system identification procedure. For control devices which cannot provide the required two states of additional stiffness to a structure like the semi-active friction dampers, the parameter identification and damage detection of the controlled structure can be performed in the time domain as long as the control forces can be measured. The conceptual design of an integrated health monitoring and vibration control system is finally performed in this thesis by taking the reticulated steel shell as an example. The main objectives of installing the integrated system are demonstrated based on the information collected and the layout of the integrated system is illustrated in detail. The main contributions and conclusions made in this thesis can be summarised as follows

1. A framework for the evaluation of potential damage due to atmospheric corrosion to steel space structures has been developed and applied to a large steel space structure and a reticulated shell built in China. The case study demonstrates the feasibility of the proposed framework. For the large steel space structure, the corrosion depth of the two steel materials increases with time.
Q235B steel has relatively smaller corrosion depth than Q345A steel. The atmospheric corrosion rate of the material depth is almost the same for the two materials and the corrosion rate of material depth is much faster in the first 5 years than later. The case study also shows that the changes in natural frequencies of the steel space structure due to either the inner surface corrosion or double surface corrosion are very small. Even though both the inner and outer surface corrosion are considered, the maximum change in the second natural frequency is about 2% in 20 years. Therefore, it can be concluded that the natural frequencies of the large steel space structure considered in this study are only slightly affected by the atmospheric corrosion of materials. Though the stresses in most of the structural members are only slightly affected by atmospheric corrosion, a few of the structural members have large stress change and such a stress change increases with increasing corrosion year. While both the natural frequency and stress changes in the reticulated shell due to atmospheric corrosion are quite small. These observations are different to the results of the large steel space structure discussed in Chapter 3 to some extent. The proposed framework for evaluation of potential damage due to atmospheric corrosion can be used to perform assessment case by case.

2. The research work on atmospheric corrosion of steel space structures is then extended by involving stress corrosion cracking to estimate corrosion damage to steel space structures in a more realistic way. An evaluation method for coupled atmospheric corrosion and stress corrosion cracking of steel space structures is presented in consideration of different locations and shapes of initial cracks as well as different periods of atmospheric corrosion. The proposed method is
applied to the large steel space structure to evaluate its potential corrosion
damage. Based on the analytical results of atmospheric corrosion and stress
corrosion cracking and the sensory technology, a corrosion monitoring system is
conceptually designed to monitor the large steel space structure in corrosive
environment and to update the proposed evaluation model, which will also form
a sub-system of the integrated health monitoring and vibration control system for
the reticulated steel shell in the last phase of this study. Numerical investigation
demonstrates that the corrosion damage of large steel space structure under dead
loads will not cause the fracture of structural members while several components
of truss girders close to concrete towers beyond the warning level of SCC. The
extreme circumferential cracks are more dangerous than the semi-elliptical
cracks at both member body and connection joints. The measured environmental
information reveals that the rapid expansion of city scale and population
substantially affect the city climate and increase the atmospheric contaminants
which may increase the corrosion damage of steel space structure. The
monitoring system for corrosion damage is conceived as an environment-based
system involving the techniques of system analysis, safety evaluation, field
inspection and data acquisition technology to monitor the structural performance
in corrosive environment and develop knowledge of monitoring methods of steel
space structures. This may advance the application of corrosion science,
structural analysis and measurement technology to operation and maintenance of
civil engineering structures. Several databases are constructed to record the
information of climate, atmospheric contaminants, strain responses, material
corrosion and crack. The sensor arrangement for the large steel space structure is
illustrated to achieve the goals of monitoring climate change, atmospheric
contaminants, material corrosion and member responses and evaluating corrosion damages. The risks of coupled atmospheric corrosion and SCC are analyzed to provide the guidelines for the safety evaluation of the reticulated shell. The presented framework can also be utilized for corrosion damage assessment and monitoring of other steel space structures.

3. The features of signal discontinuity in acceleration response time histories of civil engineering structures due to sudden stiffness reduction have been examined and an instantaneous damage index has been proposed. Two damage detection approaches in terms of the proposed damage index have been put forward for the online and offline detection, respectively, of damage time instant, damage location, and damage severity. The two proposed detection approaches can accurately identify the damage time instant and damage location due to a sudden stiffness reduction of the structure in terms of the occurrence time and spatial distribution of sharp damage indices. The damage index is linearly proportional to damage severity but the slope of linear function depends on external excitation and damage time instant. The two proposed detection approaches are applicable to both the shear building and the reticulated shell. The approach without using EMD is suitable for online health monitoring and damage detection systems. The two approaches can identify the damage time instant and damage location from the contaminated acceleration responses of the structures if the upper bound of noise frequency range is low. If the noise frequency range is wide enough, the reliability of damage detection using the proposed approaches significantly deteriorates with the increase of noise intensity. As long as the damage event can be identified, the magnitude of
damage index remains almost the same for different noise intensities and frequency ranges no matter which approach is used. For the sudden reduction of axial stiffness of shell members, only one accelerometer is required for a member to monitor the possible sudden stiffness reduction because the damage information fully exists in the acceleration responses in the member’s axial direction.

4. An integrated procedure for health monitoring and vibration control of a building using semi-active friction dampers has been developed. In such an integrated approach, a model updating scheme based on adding known stiffness by using semi-active friction dampers is first presented to update the structural stiffness and mass matrices and to identify its structural parameters using measured modal information. Based on the updated system matrices, the control performance of semi-active friction dampers with a given control algorithm is then investigated for either the building or the shell against earthquakes. By assuming that the building suffers certain damage after an extreme event or long-term service and by using the previously identified original structural parameters, a damage detection scheme based on adding known stiffness using semi-active friction dampers is proposed and used for damage detection. The feasibility and effectiveness of the proposed integrated procedure are demonstrated through detailed numerical investigation on the shear building. The numerical results from model updating and system identification clearly demonstrate that the identification of structural parameters first and the construction of stiffness and mass matrices afterwards using the proposed model updating scheme are more feasible and accurate than the existing method of directly updating the stiffness
and mass matrices. If the information of input excitation can be obtained and the natural frequencies and mode shapes are identified using the known input excitation, the identification quality of the structural parameters using the proposed scheme could be enhanced significantly. The numerical results from seismic response control demonstrate that the control performance of the local control strategy with the Kalman filter is similar to that using the local control strategy without the Kalman filter. Because the sensory system required by the local control strategy with the Kalman filter is accelerometers only, it is thus superior to the local control strategy without the Kalman filter for the integrated health monitoring and vibration control. The numerical results from damage detection demonstrate that the quality of damage detection using the proposed scheme is similar to that using the traditional sensitivity-based approach.

5. The integrated procedure for health monitoring and vibration control using semi-active friction dampers proposed for the shear building is also applied to the reticulated shell. The seismic responses of the reticulated shell without control are examined to explore the dangerous parts of the shell under earthquake. Based on these observations, two damper installation schemes are presented to determine the effective damper placement for satisfactory control performance. For vibration control, the equation of motion of controlled reticulated shell is deduced following the finite element theory. A local control strategy only using the local information of friction dampers is utilized to realize the seismic mitigation. For system identification, the parameter identification scheme based on adding known stiffness proposed for shear building is extended for the reticulated shell. The transform matrix of stiffness parameters are deduced based
on the structural connectivity and transformation information. In addition, the damage detection method proposed for shear building is also extended for the reticulated shell. The numerical results demonstrate that the dynamic responses of the reticulated shell can be effectively suppressed using the semi-active friction dampers based on local control strategy with fixed increment of slipping force. The damper installation scheme No.2 presents better control performance than that of scheme No.1 and the scheme No.2 therefore is adopted to provide known stiffness for parameter identification and damage detection. The numerical results from parameter identification and damage detection clearly demonstrate that the identification results are accurate without noise contamination. The identification accuracy reduces quickly with the increase of noise intensity. The stiffness parameters and damage events can be identified only under noise intensity no more than 0.1%. The numerical results demonstrate that the quality of parameter identification and damage detection for the reticulated shell is inferior to that of the shear building under the same noise intensity.

6. The parameter identification and damage detection of the controlled structures are numerically investigated in the time domain. The equation of motion of the controlled structure is first converted to the parameter identification equation when the inertia forces, damping forces, and restoring forces are linear functions of structural parameters. By taking control forces as known external forces together with the measured structural responses, the least-squares method adopting an amplitude-selective filter is then used to solve the parameter identification equation, from which the structural parameters can be identified.
The same procedure is applied to the controlled structure with damage to identify another set of structural parameters. By comparing the two sets of structural parameters identified, the structural damage can finally be detected and quantified. This proposed procedure is applied to the shear building and the reticulated steel shell with control devices for parameter identification and damage detection with and without measurement noise. The identification results demonstrate that all the stiffness parameters of the shear building and the reticulated shell can be accurately identified by utilizing noise free measurement information of dynamic responses. The identification accuracy of stiffness parameters of the shear building and the reticulated shell decreases with the increase of noise contamination. To compare the identification quality with/without control forces, one can find that both of them can accurately determine the stiffness parameters of the shear building and the reticulated shell without noise contamination. With the introduction of noise contamination, the identification quality without control forces is slightly better than those with known control forces. The numerical results also demonstrate that the quality of parameter identification and damage detection for the reticulated shell is inferior to that of shear building under the same noise intensity.

7. The conceptual design of an integrated health monitoring and vibration control system is finally performed in this thesis by taking the reticulated steel shell as an example with the aim of updating analytical models, identifying structural parameters, assessing structural safety, guiding maintenance and repairing work, and activating control devices to protect the structure against extreme loading. The structural responses due to changes in ambient temperature and fire are
firstly examined. The results reveal that the nodal displacements caused by small
temperature variations (within 5 degrees Celsius) are close to those induced by
dead loads. In a fire, the quick increase of material temperature will inevitably
cause the loss of structural load bearing capacity and further lead to local
instability under dead loads which will eventually leads to the collapse of the
entire shell. Compared with earthquake-induced responses, the responses under
wind excitations are relative small and it is unnecessary to conduct wind-induced
vibration control for the reticulated shell. The structural behaviour, stability and
safety of the reticulated steel shell under dead load, wind load, earthquake load,
temperature, fire and corrosion are summarised. Based on the observations
made in various analyses, the design of integrated health monitoring and
vibration control system for the reticulated shell is executed. The objectives of
establishing the integrated system for the reticulated shell is first illustrated.
Various types of sensors are selected to measure climate change, atmospheric
contamination, material corrosion, wind, earthquake, structural responses, and
control forces among others. The numbers and locations of the sensors and
control devices are also specified. The sensory system includes many kinds of
sensors: acidimeter, anemometer, hygrometer, pyranometer, ombrometer,
temperature sensors, SO\textsubscript{2} sensor, NO\textsubscript{2} sensor, Cl\textsuperscript{-} sensor, corrosion sensor, wind
pressure sensor, seismometer, fire alarm devices, accelerometers, displacement
transducer, laser displacement transducer, strain gauge and force transducer. The
field inspection work required for structural safety assessment is illustrated. Two
databases are established to collect the information from the sensors and the
inspection respectively. Based on the collected information, the analytical
models are required to be updated for structural analysis, system identification
and safety assessment. Finally, the system operation and layout is illustrated in detail.

10.2 RECOMMENDATIONS

Although some progress has been made in this thesis to the development and application of health monitoring and vibration control techniques for evaluating the safe conditions and suppressing dynamic responses of steel space structures, there remain some important issues deserving further study to enhance our understanding of the integrated monitoring and control system and its application.

1. The crack types adopted in the assessment of corrosion damages are typical semi-elliptical crack and circumferential crack. However, in practice, the cracks exist in various irregular configurations. Therefore, it is more practical to extend the evaluation method for corrosion damages based on the crack configurations from field inspection.

2. The corrosion damage of steel space structures has been studied involving the coupled atmospheric corrosion and stress corrosion cracking. The evaluation of corrosion fatigue damage is not executed because wind-induced pressure time histories are not available and the reticulated shell is dominated mainly by compressive forces. Therefore, the assessment on corrosion fatigue damage needs further investigation for other types of steel space structures. Wind tunnel tests to determine wind pressure time histories and distributions over steel space structures are accordingly required.
3. The performance of the sudden damage detection is assessed through computation simulation based on the instantaneous index. The experimental investigation or a real application to steel space structures would enhance the understanding of the index developed in this thesis and is therefore recommended.

4. The effectiveness of the semi-active friction damper on seismic mitigation of steel space structures was assessed through numerical study. The experimental investigation on control performance of semi-active friction dampers has been conducted only for shear buildings (Ng and Xu). It is therefore necessary to experimentally examine its control performance on steel space structures.

5. In the present study, the shear building and the reticulated shell are assumed to vibrate elastically regardless of the type and magnitude of the earthquakes. It is possible that the structures may yield and thus vibrate nonlinearly under severe earthquakes. The evaluation of control performance by taking the nonlinearity of vibration into consideration needs to be studied.

6. The identification method developed for integrated health monitoring and vibration control system is only applied to the civil engineering structures installed with semi-active friction dampers. Further investigations aim to develop effective identification approaches for controlled structures with other types of controllers such as active control devices are expected.
7. The identification results for the reticulated shell with a great number of DOFs under large noise intensity are not satisfactory. It is therefore necessary to develop effective identification method for large scale structures. In addition, the identification approaches are carried out for the reticulated shell with complete input information. Further investigations for proposing effective identification methods based on incomplete input information in both frequency and time domain are necessary for large scale structures.
APPENDIX A

MODE SHAPES OF RETICULATED SHELL
$f_1 = 4.066 \ (Hz)$

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A-3

$f_0=4.483 \ (Hz)$

$f_{10}=4.483 \ (Hz)$

$f_{10}=4.541 \ (Hz)$

$f_{12}=4.541 \ (Hz)$

$f_{13}=4.561 \ (Hz)$

$f_{14}=4.594 \ (Hz)$

$f_{15}=4.617 \ (Hz)$

$f_{16}=4.695 \ (Hz)$
Figure A.1 The first twenty mode shapes of the reticulated shell
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