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MONITORING-BASED ASSESSMENT OF BRIDGES SUBJECT TO SHIP COLLISION

YANLIN GUO

M. Phil

The Hong Kong Polytechnic University

2010
CERTIFICATE OF ORIGINALITY

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_________________ (Signed)

Yanlin GUO (Name of student)
To my parents
ABSTRACT

The bridges over waterways are exposed to ship collision. The risk of ship-bridge collision is increasing due to the increase in the frequency of commercial ship trips in modern times. There are hundreds of ship-bridge collision accidents worldwide every year. Rather than causing the collapse of whole bridges, this kind of accidents usually leads to invisible structural damage, which is well hidden behind an apparent structural integrity condition and may bring about hidden danger threatening the bridge safety. As a result, post-collision condition/damage assessment for ship-collided bridges is of great importance when deciding whether to close a bridge to traffic and when planning consequent bridge retrofitting. The work described in this thesis is devoted to developing a sensor placement optimization method targeting post-collision damage detection of bridges and a set of Hilbert-Huang transform (HHT) based procedures for ship-bridge collision accident alarming and condition/damage assessment of the collided bridges using the monitoring data.

On-line structural health monitoring (SHM) systems provide a unique approach to monitor bridge responses during ship collisions and detect the structural damage. The damage information contained in the monitoring data, which is critical for damage detection, however, is largely dependent on the sensor layout. Therefore, an optimal sensor placement method targeting post-collision damage detection of bridges is proposed for selecting the optimal sensor set so that the measured data are
most informative for damage detection. The sensor configuration is optimized by a multi-objective optimization algorithm which simultaneously minimizes the information entropy index for each possible ship-bridge collision scenario. One advantage of the proposed method is that it can handle the uncertainty of ship collision position. It also guarantees a redundancy of sensors for the most informative regions, and leaves some freedom to determine the critical elements for monitoring. The proposed method is applicable in practice to determine the sensor placement, prior to field testing, with the intention of identifying post-collision damage. The cable-stayed Ting Kau Bridge in Hong Kong is employed to demonstrate the feasibility and effectiveness of the proposed method.

It is promising to detect ship-bridge collision accidents and assess post-collision condition using the monitoring data acquired by SHM systems for the instrumented bridges. The ship-collision-induced bridge dynamic responses are transient, non-stationary and possibly non-linear. In considering the superiority of HHT in representing non-linear and non-stationary signals without any artefacts imposed by the non-locally adaptive limitations of the fast Fourier transform (FFT) and wavelet processing, a set of HHT based techniques is developed for the ship-bridge collision accident detection and ship-collided bridge condition assessment by use of the monitoring data. Firstly a viable accident alarming method based on the original HHT is developed to enhance the accuracy of ship collision accident alarming. Subsequently, a hybrid HHT based approach, which combines the empirical mode decomposition (EMD), FFT, band-pass filter and Hilbert transform, is proposed to
interpret the condition of ship-collided bridges. The instantaneous frequencies and transient energy distributions of the collision-induced responses reflected by the resultant Hilbert transform are analyzed in the time-frequency domain, and are compared with the traditional power spectral densities obtained by the FFT and those by the wavelet transform. The measured acceleration responses of the suspension Jiangyin Bridge during a ship collision event are used to testify the effectiveness of the HHT based procedures in alarming the ship-bridge collision accident and assessing the ship-collided bridge condition. The results show that the intrinsic mode functions obtained by the HHT do signal the accident clearer and more accurate than the raw signals, and the hybrid HHT based approach is effective in detecting the structural damage incurred during ship collision.
LIST OF PUBLICATIONS

Journal Articles


Conference Papers


the 7th International Workshop on Structural Health Monitoring, 9-11 September 2009, Stanford, California, USA.

ACKNOWLEDGEMENTS

I would like to express my deepest gratitude to my supervisor, Prof. Y. Q. Ni, for introducing me to the world of structural health monitoring, and his enlightening guidance and advice, strong sense of responsibility, and continuous support during my studies and preparation of the thesis. Without his guidance and support, the work would never be completed. The privilege of working with Prof. Y. Q. Ni has appreciably influenced my professional development and perspectives. In addition, I would like to thank Prof. M. L. Wang, Dr. Y. Xia and Prof. S. Zhan for their instructions, suggestions and supports.

A grant from the Research Grants Council of the Hong Kong Special Administrative Region, China (Project No. PolyU 5253/06E) and a grant from The Hong Kong Polytechnic University through the Development of Niche Areas Programme (Project No. 1-BB68) are gratefully acknowledged.

I would also like to express my sincere thanks to my colleagues Dr. H. F. Zhou, Dr. Y. F. Duan, Mr. Z. H. Chen, Mr. X. W. Ye, Mr. S. K. Chen, and Mr. H. W. Xia.

Finally, I would like to express my thanks to my parents, other family members, teachers, and friends for their encouragement, trust, friendship and love during my two years study.
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
</tr>
<tr>
<td>ADPR</td>
<td>Average driving-point residue</td>
</tr>
<tr>
<td>ALARP</td>
<td>As low as reasonably practicable</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>BSI</td>
<td>British Standards Institute</td>
</tr>
<tr>
<td>DMM(s)</td>
<td>Damage monitoring model(s)</td>
</tr>
<tr>
<td>DMM1</td>
<td>Damage monitoring model 1</td>
</tr>
<tr>
<td>DMM2</td>
<td>Damage monitoring model 1</td>
</tr>
<tr>
<td>DOF(s)</td>
<td>Degree(s) of freedom</td>
</tr>
<tr>
<td>DOT</td>
<td>Texas Department of Transport</td>
</tr>
<tr>
<td>DWT</td>
<td>Dead weight tonnage</td>
</tr>
<tr>
<td>EEMD</td>
<td>Ensemble empirical mode decomposition</td>
</tr>
<tr>
<td>EI</td>
<td>Effective independence</td>
</tr>
<tr>
<td>EI-DPR</td>
<td>Effective independence driving-point residue</td>
</tr>
<tr>
<td>EMD</td>
<td>Empirical mode decomposition</td>
</tr>
<tr>
<td>EVP</td>
<td>Eigenvalue vector product</td>
</tr>
<tr>
<td>FE</td>
<td>Finite element</td>
</tr>
<tr>
<td>FFT</td>
<td>Fast Fourier transform</td>
</tr>
<tr>
<td>FHWA</td>
<td>Federal Highway Administration</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>FIM</td>
<td>Fisher information matrix</td>
</tr>
<tr>
<td>FRF(s)</td>
<td>Frequency response function(s)</td>
</tr>
<tr>
<td>GA</td>
<td>Genetic algorithm</td>
</tr>
<tr>
<td>HHT</td>
<td>Hilbert-Huang transform</td>
</tr>
<tr>
<td>IABSE</td>
<td>International Association for Bridge and Structural Engineering</td>
</tr>
<tr>
<td>IMF(s)</td>
<td>Intrinsic mode function(s)</td>
</tr>
<tr>
<td>INA</td>
<td>International Navigation Association</td>
</tr>
<tr>
<td>KE</td>
<td>Kinetic energy</td>
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<td>OSP</td>
<td>Optimal sensor placement</td>
</tr>
<tr>
<td>SA</td>
<td>Simulated annealing</td>
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<td>SCIR</td>
<td>Ship-collision-induced response</td>
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<tr>
<td>SHM</td>
<td>Structural health monitoring</td>
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<tr>
<td>SWRC(s)</td>
<td>Steel-wire-rope coil(s)</td>
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<td>TKB</td>
<td>Ting Kau Bridge</td>
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CHAPTER 1
INTRODUCTION

1.1 Research Background

1.1.1 Review of ship-bridge collision accidents

Ship collisions with bridges over waterways have become an important issue in modern times. Hadiopriono (1985) has identified collisions as a major contributor to bridge collapse through the survey of structural failures since 1985. Not only can a bridge collapse result in a loss of life, it may also cause an impasse for trains, automobiles, and commercial vehicles, resulting in a great economic loss for the community. Recently, a striking high frequency of ship bridge collision accidents in China highlights the immediate need of re-examination of the ship-bridge collision problem.

During the past two years, there have been four serious ship collision accidents happening in China, two of which led to catastrophic bridge failures. On June 15, 2007, the Highway 325 Bridge over the Jiujiang River in Foshan, Guangdong, China, collapsed due to a ship collision, with the deck of the middle span dropping into the river. The accident led to eight fatalities and four vehicles plunging into the river. On January 1, 2008, the Guoqiang Bridge in Haimen, Jiangsu, China, was struck by a ship and the keel of the bridge was damaged. On March 27, 2008, the deck of the under-construction Jintang Bridge in Ningbo, Zhejiang, China, was struck by the mast of a ship, resulting in one span of the box girder dropping into the river. Four sailors were killed in the accident. On November 16, 2009, just five days before its
opening, the Jintang Bridge encountered a second time ship-bridge collision accident, which caused the partial damage in two piers and thus deferred the date of the bridge opening to traffic. In addition, there are numerous small ship collision accidents with bridges every year which cause structural damage in different degrees, but do not necessarily result in collapse of the structures. There have been more than 70 ship collisions with the Wuhan Yangtze River Bridge since its opening in 1957 and about 30 ship collisions with the Nanjing Yangtze River Bridge since its opening in 1968 (Dai et al. 2002; Xiang et al. 2002). More than 300 incidents on ship collision with the bridges on the Yangtze, Pearl, and Heilongjiang Rivers have been recorded (Dai et al. 2002).

In the USA, according to a continuous study in bridge failures undertaken from 1951 to 2000, 15.8% of the 114 bridge failures were caused by ship collision from 1951 to 1988 (Harik et al. 1990); between 1989 and 2000, 10 bridges out of the 503 collapsed bridges were damaged by ship collision (Wardhana and Hadipriono 2003).

From a global perspective, a comprehensive review of 150 ship-bridge collision incidents from 1960 to 1998 conducted by Van Manen and Frandsen (1998) showed that the rate of very serious collisions steadily increased from 1960 to 1980, however, from 1980 a decrease in collision rate was observed due to a special attention paid to the problem in that period. Unfortunately, an increase in collision rate has been noticed again from 1990s (Van Manen and Frandsen 1998). The serious fatal bridge failures due to ship collision which claimed lives are illustrated in Table 1.1. Between 1960 and 2008, the total loss of lives due to ship-bridge collision was 355.
## Table 1.1 Fatalities in bridge failures due to ship collision (adapted from Proske and Curbach (2005))

<table>
<thead>
<tr>
<th>Bridge name</th>
<th>Year</th>
<th>Number of deaths</th>
</tr>
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<tr>
<td>Severn River Railway Bridge, UK</td>
<td>1960</td>
<td>5</td>
</tr>
<tr>
<td>Lake Ponchartrain, USA</td>
<td>1964</td>
<td>6</td>
</tr>
<tr>
<td>Sidney Lanier Bridge, USA</td>
<td>1972</td>
<td>10</td>
</tr>
<tr>
<td>Lake Ponchartrain Bridge, USA</td>
<td>1974</td>
<td>3</td>
</tr>
<tr>
<td>Tasman Bridge, Australia</td>
<td>1975</td>
<td>15</td>
</tr>
<tr>
<td>Pass Manchac Bridge, USA</td>
<td>1976</td>
<td>1</td>
</tr>
<tr>
<td>Tiorn Bridge, Sweden</td>
<td>1980</td>
<td>8</td>
</tr>
<tr>
<td>Sunshine Skyway Bridge, USA</td>
<td>1980</td>
<td>35</td>
</tr>
<tr>
<td>Lorraine Pipeline Bridge, France</td>
<td>1982</td>
<td>7</td>
</tr>
<tr>
<td>Sentosa Aerial Tramway, China</td>
<td>1983</td>
<td>7</td>
</tr>
<tr>
<td>Volga River Railroad Bridge, Russia</td>
<td>1983</td>
<td>176</td>
</tr>
<tr>
<td>Claiborn Avenue Bridge, USA</td>
<td>1993</td>
<td>1</td>
</tr>
<tr>
<td>CSX/Amtrak Railroad Bridge, USA</td>
<td>1993</td>
<td>47</td>
</tr>
<tr>
<td>Queen Isabella Memorial Bridge, USA</td>
<td>2001</td>
<td>8</td>
</tr>
<tr>
<td>Webber-Falls, USA</td>
<td>2002</td>
<td>14</td>
</tr>
<tr>
<td>Highway 325 Bridge over the Jiujiang River, China</td>
<td>2007</td>
<td>8</td>
</tr>
<tr>
<td>Jintang Bridge, China</td>
<td>2008</td>
<td>4</td>
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Based on the intensive review of ship-bridge collision accidents, the two main reasons for the high frequency of the problem can be traced as follows.

Firstly, ship collision accidents are inevitable. Frandsen (1983) conducted an extensive survey of bridges that collapsed due to ship collision. Over half of these accidents were due to human error by the ship’s master or pilot. Around a quarter occurred because of steering failure and the remainder due to ships breaking loose from moorings during storms. All such hazards cannot be prevented and must be considered in design (Shiraishi and Cranston 1992).

Secondly, in modern times, the technology advances increase the potential risk on ship-bridge collision. With the development of bridge design and construction techniques, more and more bridges, especially those with long spans and deep foundations, have been constructed across navigation waterways. From 1960 to 1997, the number of U.S. bridges that cross major waterways at coastal ports increased 33% (Mastaglio 1997). The construction of these bridges has made the navigation environment complicated and thus has increased the difficulty for ship navigating. On the other hand, the advances in shipping techniques have led to the increasing frequency of commercial ship trips and the existence of ships with large size and heavy dead weight tonnage (DWT). The number of vessels in the world fleet has increased threefold since 1960. Today the ships can reach a length of up to 200 m, a breadth of up to 12 m and a weight of several thousand tons (Proske and Curbach 2005). All of these technology advances not only result in an increasing frequency of shipping trips under the bridges, which means an increasing risk on collision
accidents, but also make the potential ship-bridge collision induced consequences be very serious.

1.1.2 Briefing of research on ship-bridge collision problem

The collapse of the original Sunshine Skyway Bridge in 1980 was a major turning point in awareness and increased concern for the safety of bridges crossing navigable waterways. Since then, research into the ship-collision problem has initiated in many countries over the world, in connection with the evaluation of vulnerability of existing bridges, the establishment of design criteria for new bridges, and the development of codes and specifications regarding vessel-bridge collisions (Larsen 1993). In 1983 the first international symposium on the topic “Ship collision with bridges and offshore structures” under the auspices of the International Association for Bridge and Structural Engineering (IABSE) took place in Copenhagen, Denmark. Important steps in the development of modern ship collision design principles and specifications have been made since then (Knott and Pruca 2000). The following two major events were the publications of the Guide Specification and Commentary for Ship Collision Design of Highway Bridges by the American Association of State Highway and Transportation Officials (AASHTO 1991) and an IABSE document entitled “Ship collision with bridges: the interaction between vessel traffic and bridge structures” (Larsen 1993). The two publications provide information and the most significant improvement of vessel impact influence on bridges. The AASHTO guide specification (AASHTO 1991) presented information and methods for conducting an evaluation, but it did not require a vessel
impact analysis as part of new bridge design (McClelland 2005). Later, AASHTO *LRFD Bridge Design Specifications and Commentary* (AASHTO 1994) incorporates all of the analysis and design requirements of the AASHTO guide specification (AASHTO 1991). Unlike the guide specification, whose usage is optional, the vessel collision requirements are mandatory in LRFD code. After that, an important step in this topic was the establishment of an international database for ship collision accidents by the PLANC Working Group. This database includes 150 collision scenarios including all the catastrophic events and also many minor accidents from 1960 (Van Manen and Frandsen 1998; INA 2001). The database is quite valuable in establishing models of ship behavior with regard to bridges, especially the probabilistic models.

### 1.2 Research Motivation

Although engineering technology dealing with ship-bridge collision problem has been improved during these years, it is still in its infant stage compared with earthquake engineering. While dynamic analysis has been applied in earthquake engineering, the present engineering technology for ship impact is often based on simple static and linear structural analysis, which is too simple and unsatisfactory. Several aspects such as the collision loads to be used in design, and the appropriate combination of extreme events (such as ship collision plus scour) have not been well understood yet (Knott and Pruca 2000). While an international database recording the collision events worldwide has been established (Van Manen and Frandsen 1998; INA 2001), the measurement of ship impact loading and bridge dynamic response
during a collision accident was never reported.

Thanks to the recent development of structural health monitoring (SHM) systems, it is possible to measure the dynamic responses of bridges during a collision event. In March, 2005, during the superstructure erection period, a barge rammed the Sutong Bridge piers and the structural responses during the vessel collision have been measured by the sensors embedded inside the piles and pile caps. In June, 2005, shortly after the monitoring system commenced to operate, the upper portion of a pile-driving boat struck the Jianyang Bridge deck and the structural responses during the ship-bridge collision accident have been collected by various sensors including accelerometers, displacement transducers, optical fiber strain sensors, load pins and elasto-magnetic sensors (Ko et al. 2005). The measured data of bridge responses/deformations during ship collision provide unique information for post-collision bridge evaluation and retrofitting. This highlights the necessity of developing methodologies of post-collision damage and condition assessment for instrumented bridges.

1.3 Research Objectives

This MPhil study aims to develop specific methods for assessing the damage and safety of bridges after experiencing ship collision. Most of ship collision incidents only cause partial damage of a bridge rather than collapse of the whole structure (Larsen 1993; INA 2001). As a result, assessment of structural damage and change in structural condition immediately after a ship collision event is very important when deciding whether to close a bridge to traffic and when planning
consequent bridge retrofitting. An on-line SHM system makes it feasible to detect post-collision damage of the instrumented bridge using monitoring data. The research proposed in this MPhil study is devoted to:

(i) Development of a sensor placement optimization method targeting post-collision damage detection;

(ii) Development of a set of approaches for ship-bridge collision accident alarming and condition/damage assessment of collided bridges using Hilbert-Huang transform (HHT) based techniques.

1.4 Thesis Outline

This thesis comprises five chapters, and is organized as follows.

Chapter 1 introduces the background and motivation for the proposed research and expounds the objectives to be pursued in this MPhil project.

Chapter 2 contains a review of the literature regarding the ship-bridge collision problem on four topics: ship impact forces, risk analysis, protection measures, surveillance and warning systems.

Chapter 3 focuses on the development of a sensor placement optimization method targeting post-collision damage detection of bridges subject to ship collision. Firstly the research need is identified, the literature on sensor place optimization is reviewed, and the limitations of the existing methods are discussed. Then the principle of the proposed method is presented. The sensor configuration is optimized by a multi-objective optimization algorithm which simultaneously minimizes the information entropy index for each possible ship-bridge collision scenario. The
cable-stayed Ting Kau Bridge (TKB) in Hong Kong is employed to demonstrate the feasibility and effectiveness of the proposed method. In the end, the advantages of the method are discussed.

In Chapter 4, a set of approaches for ship-bridge collision accident alarming and condition/damage assessment of collided bridges using HHT based techniques are developed. At first, a literature review on signal analysis based techniques for structural condition assessment is provided, and the limitations of the existing techniques are identified. Then the motivation of this study is introduced. Subsequently, a viable accident alarming method based on the original HHT is developed to enhance the accuracy of ship collision accident alarming and a hybrid HHT based method, which combines the empirical mode decomposition (EMD), fast Fourier transform (FFT), band-pass filter and Hilbert transform, is proposed to interpret the condition of ship-collided bridges. The measured acceleration responses of the suspension Jiangyin Bridge during a ship collision event are used to testify the effectiveness of the HHT based approaches in alarming the ship-bridge collision accident and assessing the ship-collided bridge condition. The instantaneous frequencies and transient energy distributions of the collision-induced responses reflected by the resultant Hilbert transform are analyzed in the time-frequency domain, and are compared with the traditional power spectral densities obtained by the FFT and those by the wavelet transform.

Chapter 5 summarizes the contributions, findings and conclusions achieved in this MPhil project. Recommendations for future work are also presented.
CHAPTER 2
LITERATURE REVIEW

2.1 Ship Impact Forces

Ship impact forces during ship-bridge collision events are foundational element that should be considered, when designing a new bridge and when evaluating an existing bridge over waterways. However, the determination of the ship impact forces during a ship-bridge collision accident is very complex, since ship impact forces are dependent on numerous factors, such as the time, the vessel type, size, velocity, the loading condition and degree of water ballast, the location and direction of impact, as well as the geometry, strength, stiffness, ductility and redundancy characteristics of the pier, and the effectiveness of the existing protection system (Prucz and Conway 1999). Many scholars have been involved in the researches on the estimation of ship impact forces in the past five decades. Mainly, these investigators are based on collision energy analysis and finite element (FE) analysis which are reviewed in the following Sections 2.1.1 and 2.1.2, respectively. In each of the two sections, impact forces estimation for ships navigating in seaways and barges navigating in inland waterways are discussed in two sub-sections, respectively.

2.1.1 Determination of impact forces through collision energy analysis

2.1.1.1 Ship impact

The majority of the literature on ship collision in the early days investigated the ship impact forces based on the analysis of collision energy transfer during the collision process and aimed at establishing the relationship between the impact force
and the indentation (damage depth).

Figure 2.1 Linear relationship between deformed vessel volume and absorbed impact energy
(Svensson 2006)

A pioneer energy model was established by Minorsky in 1959, based on 26 ship-ship collision cases (Svensson 2006). In this model, a linear relationship between the deformed vessel volume and the absorbed impact energy is established, as shown in Figure 2.1. Based on this relationship, Woisin (1976) developed an equation giving the average collision force, which can be used as design impact load directly:

\[
P_0 = \frac{V^{2/3} \cdot L^2}{1100}
\]

(2.1)

where:

- \( P_0 \) is the average collision impact load (MN);
- \( V \) is the vessel velocity (m/sec);
- \( L \) is the vessel’s length (m).
This equation was later used in the design of the Bahrain Causeway Bridge and Faroe Bridge (Larsen 1993).

However, the Minorsky’s relationship is based on the collision between two ships, it may not be accurate enough to define the impact load in bridge design. The necessity to define a more accurate design impact load pushed the subsequent researchers to investigate the impact forces during ship-bridge collisions. The ideal case of a ship collision with a rigid wall is then assumed in most researches in 1960s and 1970s. Dynamic model tests ranging from low energy to high energy were widely carried out and culminated in the publication of a force-indentation relationship and a force history curve (given in Figure 2.2) by Woisin (Svensson 2006). In Figure 2.2, the average impact force is estimated about half of the maximum impact force. Woisin concluded from the test results that the equivalent static impact forces are proportional to the square root of DWT of the ship. Considering the existence of a scatter for the ships with the same DWT due to the

![Figure 2.2 Dynamic relationship between ship impact force and time (Svensson 2006)](image-url)

Figure 2.2 Dynamic relationship between ship impact force and time (Svensson 2006)
bow construction and collision speeds, Saul and Svensson (1983) made a modification of Woisin’s approach and proposed an effective maximum impact force formula as follows.

\[ P_{\text{max}} = 0.88(DWT)^{1/2} \pm 50\% \]  \hspace{1cm} (2.2)

For design purposes, Knott (1998) suggested that 70% fractile of average collision force should be used as equivalent static vessel impact force in design, as shown in Figure 2.3.

![Figure 2.3 Probability density function of ship impact force (Knott 1998)](image)

Considering the influence of vessel speeds, the AASHTO provisions use the following formula to estimate the static head-on ship collision force (AASHTO 1991):

\[ P_\ell = 0.98(DWT)^{1/2}(V/16) \]  \hspace{1cm} (2.3)

In the early 1990s, a different statement with the previous study in defining the
design equivalent static impact loads was proposed by Pedersen et al. (1993). They developed a new force-indentation relationship, indicating that the impact force is gradually increasing during the whole course of the collision accident. According to the results, they suggested that the maximum impact force rather than the average force should be used in design when the design method is based on static force analysis.

Although the ship impact forces are of dynamic characteristics, most codes and specifications use the equivalent static force as design force. The Eurocode published in 2006 introduces dynamic design, considering dynamic effects of impact in design for the first time (BSI 2006). The maximum dynamic interaction force is given by the following formula:

\[ F = v_r \sqrt{km} \]  

(2.4)

where:

- \( v_r \) is the object velocity at impact;
- \( k \) is the equivalent elastic stiffness of the object;
- \( m \) is the mass of the colliding object.

The dynamic impact force is considered as a rectangular pulse acting on the surface of the structure, as shown in Figure 2.4.
2.1.1.2 Barge impact

Regarding the bridges crossing inland waterways, barge vessels, which have different shapes and structures compared to ship vessels, should be considered in defining the design vessel load. The most comprehensive studies were conducted by Meier-Dörnberg; the model developed by him was used as the basis for AASHTO specifications (Knott and Larsen 1990; AASHTO 1991; Larsen 1993; AASHTO 1994). The barge collision load recommended by AASHTO for the design of piers is
a function of the tow length and the impact speed, as shown in Figure 2.5. Numerical formulations for deriving these relationships are given in AASHTO (1991; 1994). The loads shown in Figure 2.5 were computed using a standard 59.5×10.7 m hopper barge. The impact force recommended for barges larger than the standard hopper barge is determined by increasing the standard barge impact force by the ratio of the width of the wider barge to the width of the standard hopper barge (Knott and Pruca 2000). However, Whitney et al. (1996) pointed out that the AASHTO formulas probably give unrealistic results since they are based on impact tests conducted on individual European barges. The energy dissipation due to crushing and friction between barges and the dynamic load magnification effect are neglected.

### 2.1.2 Determination of impact forces through FE analysis

In the past, the limitation of the computer technology and complex collision mechanics (including buckling, crushing, tearing of different parts of the structural members of the ship) made simulating the ship-bridge collision behavior with FE model both time consuming and difficult. However, with the improvement of the computer technology in the 21st century, it is possible to overcome some of the above problems and to simulate the ship-collision process with the consideration of both dynamic behavior and nonlinear structural behavior via FE model. Some researchers have made attempts to deal with both ship impact and barge impact and shown that the FE method has obvious advantages compared with other methods in that the structural internal stress distribution, collision force change, energy exchange, and structural deformation can be perfectly depicted by this method (Liu
and Gu 2002).

### 2.1.2.1 Ship impact

The interaction between the deformed structural components can be considered by defining master and slave surfaces in FE method. Liu and Gu (2002) simulated a scenario of a 40,000 DWT oil tanker colliding with a bridge and found that the curve of collision force is nonlinear and fluctuant, indicating continuous unloading due to structural failure and ruptures of the ship hull during collision. They also concluded that the damage to the bridge pier depends on the maximum collision force. The results may provide a basis for determining the design vessel load.

![Figure 2.6 Collision forces versus time (Wang et al. 2006)](image)

The researchers in the Tongji University, Shanghai, China, conducted a simulation of collision between a ship’s bow and a concrete bridge pier (Wang et al. 2006). The dynamic impact forces for a head-on collision of a 50,000 ton bulk carrier against a rigid wall versus time are calculated, as shown in Figure 2.6. For the
sake of comparison, the different equivalent design loads suggested by Pedersen, AASHTO and Eurocode are also shown in Figure 2.6. It is observed that the AASHTO and Eurocode equivalent loads appear to lie on the safe side, while the one suggested by Pedersen is about double of these values.

A problem when using the equivalent load method is that the equivalent load in codes determined by energy analysis is based on the assumption of ship collision with a rigid wall, which neglects the energy consumption by deformation of the pile and thus may result in a larger impact force than the one in the real condition. In order to investigate this problem, the researchers in the Tongji University compared the dynamic impact force histories for a head-on collision of a 50,000 ton bulk carrier against a rigid wall and against an elastic pile cap respectively, as shown in Figure 2.7. In this figure, the peak value of the collision force against the elastic foundation on piles is about 50% of the one against the rigid wall. Therefore, it is
questionable to determine the equivalent forces based on ship-rigid-wall collision assumption, which will lead to a conservative design.

Obviously, there still exist some controversial issues on how to define the design impact loads. For example, which value of impact force should be used, the absolute peak value, the local mean peak or the global mean value; what assumption should be used, the ship-rigid-wall collision assumption or ship-elastic-pile collision assumption. Therefore, more research effects on defining design impact forces are needed.

2.1.2.2 Barge impact

Considering the non-linear inelastic material behavior, part-to-part contact, self-contact, and large deformations during the collision, Consolazio and Cowan (2003) conducted a non-linear high-resolution FE simulation of barge-pier impact. Compared with the AASHTO barge impact force model, which does not consider the effect of pier geometry and assumes that the equivalent static impact force can be uniquely correlated to peak crush deformation sustained by a barge during an impact event, the simulation results revealed some new findings: the pier geometry (shape and size) can influence the crush behavior of a barge; impact forces do not necessarily increase monotonically and thus cannot necessarily be uniquely correlated to the maximum sustained deformation.

2.2 Risk Analysis

Risk analysis is a systematic, quantitative approach for assessing the accidental probabilities and consequences of engineering systems. It is a useful tool to enable
the designer to make rational decisions in the design for accidental loads (vessel collision, scour, earthquake, etc.). The collision risk is defined as the potential realization of unwanted consequences of ship-bridge collision event (Lee et al. 2006). The formal definition of risk combines the aspects of frequency (or how often the hazard manifests itself) and consequences (or how much harm will be done) (Milloy 1998). It is not feasible to design all parts of a bridge structure to withstand the worst case loads from ship impact. The practical method is to use risk analysis to achieve an acceptable risk level for the society. Risk analysis is used for both designing new bridges and evaluating existing bridges with regard to ship-bridge collision problems. It generally consists of two major aspects: the estimation of the risk of a bridge and determination of the risk acceptance criteria. The estimated risk is then compared with the risk acceptance criteria. From the design point of view, the bridge characteristics would be adjusted or the risk reduction requirements would be implemented until the risk acceptance is satisfied (Knott 1998). When it comes to evaluating the vulnerability of an existing bridge, the assessment can be obtained directly from the comparison results; then risk reduction measures would be designed if necessary. The two aspects of risk analysis, the estimation of the risk of a bridge and determination of the risk acceptance criteria, are introduced in Sections 2.2.1 and 2.2.2, respectively.

### 2.2.1 Mathematical risk models

The estimation of risk is realized by developing the mathematical risk models. The risk model proposed by Larsen (1993) is
\[
F = \sum N_i \cdot P_{c,i} \cdot \sum P_{G,i,k} \cdot P_{F,i,k}
\]  

(2.5)

where:

- \(P_{c,i}\) is the causation probability;
- \(P_{G,i,k}\) is the geometrical probability;
- \(P_{F,i,k}\) is the failure probability.

Based on the principle of Larsen’s model, AASHTO (1991) gives a risk model as follows.

\[
AF = (N)(PA)(PG)(PC)
\]  

(2.6)

where:

- \(AF\) is the annual frequency of bridge pier collapse;
- \(N\) is the annual number of vessels in the waterway which can strike the pier;
- \(PA\) is the probability of vessel aberrancy;
- \(PG\) is the geometric probability of a collision;
- \(PC\) is the probability of collapse.

The risk model given by Eurocode (BSI 2006) is

\[
P_f = N \int F_{\text{dyn}}(x) > R dx
\]  

(2.7)

where:

- \(N = n\lambda T(1 - P_a)\) is the total number of incidents in the period of consideration;
- \(n\) is the number of ships per time unit (traffic intensity);
- \(\lambda\) is the probability of a failure per unit traveling distance;
- \(T\) is the reference period (usually 1 year);
- \(P_a\) is the probability that a collision is avoided by human intervention;
\( x \) is the coordinate of the point of the fatal error or mechanical failure;

\( F_{\text{dyn}} \) is the impact force on the structure obtained from impact analysis;

\( R \) is the resistance of the structure.

Where relevant, the distribution of the initial ship position in the \( y \)-direction may be taken into account, referring to Figure 2.8 (BSI 2006).

![Figure 2.8 Ship collision scenario (BSI 2006)](image)

**2.2.2 Risk acceptance criteria**

The risk acceptance criteria are predetermined based on the maximum tolerance of the risk by the society. When predetermining the acceptance criteria, both the frequency describing how often the hazard manifests itself and the consequences describing how much harm will be done should be taken into consideration. It is qualitatively believed that people will have an equal tolerance to the minor accidents with a high frequency and the serious accidents with a relatively low frequency. The frequency and consequences of risk are often represented by the so-called FN
diagrams, as shown in Figure 2.9 (Milloy 1998). The risks falling into the top right area are those with both high frequency and serious consequence and cannot be tolerated, while the risks in the low left part can be tolerated by the society. The central region known as “as low as reasonably practicable (ALARP)” region indicates that further risk reduction measures should be applied (Milloy 1998). A certain risk reduction measure should be implemented if the cost of each statistical life saved is perceived to be good value by using it. However, the “good value” is somewhat of subjective judgment and always influenced by politics. Hence, it is important to remember that while risk estimates are essentially technical, the judgments regarding the acceptability of risk are essentially political (Milloy 1998).

Although the FN diagrams approach and the related ALARP approach are risk analysis tools, in practice, many governments simply consider the consequences of the vessel bridge collision accidents according to the importance of the bridges by
grouping the bridges into critical ones and regular ones and suggest the risk acceptance criteria for each group in codes or specifications. The most influential acceptance vessel collision risk criteria are recommended by AASHTO (1991). For critical bridges, the maximum annual frequency of collapse for the whole bridge shall be taken as 0.0001. For regular bridges, the maximum annual frequency of collapse for the whole bridge shall be taken as 0.001. Although the codes are widely used in many projects, there still exist some controversial comments on the criteria (Knott 1998). In the USA, it is generally believed that the ship collision risk return year may be too high and should be reduced to the 475 and 2,500 years return period levels for regular bridges and critical bridges respectively, as used for earthquake design. However, on the contrary, it is regarded in Europe that the vessel collision risk levels are too low and should be increased to 10,000 years for regular bridges, and as high as 1,000,000 years for critical bridges. This controversy results from different societies’ perceptions about the risk and financial abilities to reduce the risk. Simply speaking, determining the risk acceptance criteria is based on the society’s willingness to pay for the risk reduction.

Although the AASHTO bridge design specification stipulates the acceptance criteria and suggests that the acceptance annual frequency of collapse for an entire bridge structure should be distributed to every component susceptible to vessel collision, how to distribute the acceptance criteria over the bridge components is not recommended by the specification and is just based on subjective judgment (AASHTO 1991). Whitney et al. (1996) recommended that the annual frequency of
collapse should be distributed to each pier based on its percentage value of the replacement cost of the structure and the summation of the annual frequencies of collapse for all barge size categories, with respect to the individual piers, should be less than or equal to the annual frequency assigned to each component. Lee et al. (2006) investigated three distribution methods of the annual frequency, i.e. main pier only distribution, uniform distribution over all the components and variable distribution which means the weighted bridge acceptance criterion will be allocated to each component. The numerical examples showed that the variable distribution type based on the total bridge reliability apportionment to its piers is more practicable (Lee et al. 2006). This distribution method of risk acceptance criterion was expected to help designers make judgment more rationally.

2.3 Protection Measures

The protective works can be designed to reduce the consequence of ship-bridge collision to non-destructive levels. The ideal objective of protective works is to protect not only the bridge structure, but also the involved vessel and environment (Conway 1998). The application of protection measures is based on the cost-effective analysis. However, it is best to eliminate the need of protection works at all through optimizing the location, span length and geometry of the bridge. The cost of longer span or larger pier should be compared with the construction cost and maintenance cost of protection works. Only when the latter is smaller than the former, the protection works should be applied.
2.3.1 Typical protection systems

2.3.1.1 Artificial island protection systems

Artificial islands consist of a sand or rock core which is protected by outer layers of heavy rock armor to provide protection against wave, current and ice action (Larsen 1993). The bridge piles are protected from physical contact with the impact ship and most energy of the ship impact is absorbed by island so that the ship impact force transmitted through the island to the bridge pier can be reduced to the level lower than the lateral capacity of the pier and the pier foundation.

2.3.1.2 Floating protection systems

There are various types of floating protection systems, including cable net systems, anchored pontoons, and floating shear booms (Larsen 1993). In the cable net system, one end of the cable net is fixed on the buoys and the other end is anchored to the bottom of waterway. This system is placed in front of bridge piers, the aberrant ship will be stopped and trapped by the net. For the anchored pontoon or floating shear booms system, the larger floating pontoons or booms are anchored to the waterway bottom in front of piers. The aberrant ship will be deflected away from the pier during ship collision. The impact energy from an aberrant ship will be absorbed by the deformation of floating pontoons or booms.

2.3.1.3 Dolphin protection systems

Dolphins are typically circular cells which are constructed by driven steel sheet pilling, filled with rock, sand or concrete, and then topped by a thick concrete cap (Larsen 1993). Dolphins may also be erected by pre-cast concrete. Dolphins resist
the low energy barge tow collision by sheer mass, but in the high energy ship collision, tend to rotate, deform, and often burst under the impact forces (Conway 1998).

2.3.1.4 Pile supported protection systems

Pile groups connected together by rigid caps, free standing piles. The piles connected by relatively flexible caps may be used for protection to resist vessel impact forces. The pile groups may consist of vertical piles, which primarily absorb energy by bending, or batter piles which absorb energy by compression and bending (Larsen 1993).

2.3.1.5 Fender protection systems

Fender protection systems can be classified as timber fenders, rubber fenders, concrete fenders and steel fenders according to the construction material. For timber fenders, the energy is absorbed by elastic deformation and crushing of timber members; for rubber fenders, the energy is absorbed through elastic deformation of rubber elements either in compression, bending, shear deformations, or a combination of all three; for concrete fenders, the energy is absorbed by buckling and crushing of the concrete walls composing the fender system; for steel fenders, the energy is absorbed by compression, bending, and buckling of steel elements in the fender (Matsuzaki and Jin 1983).

2.3.1.6 Comparison of typical protection systems

The characteristics of the aforementioned five typical protection systems are compared in Table 2.1.
<table>
<thead>
<tr>
<th>Type of Protection System</th>
<th>Application Scopes</th>
<th>Advantages</th>
<th>Disadvantages</th>
<th>Costs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial island protection systems</td>
<td>• Estuarine case</td>
<td>• Very large kinetic energy absorption ability</td>
<td>• Impediment to flood or tidal flow</td>
<td>• High construction costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Limitation of the vessel damage</td>
<td></td>
<td>• Low restoration costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>• Low long term maintenance costs</td>
</tr>
<tr>
<td>Floating protection systems</td>
<td>• Very deep water</td>
<td>• Very large kinetic energy absorption ability</td>
<td>• Large space occupation around the pier</td>
<td>• Moderate high restoration costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Limitation of the vessel damage</td>
<td>• Vulnerability of corrosion</td>
<td>• High long term maintenance costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Feasibility in very deep water</td>
<td>• Problematic durability in waters subject to icing or ice drift</td>
<td></td>
</tr>
<tr>
<td>Dolphin protection systems</td>
<td>• Estuary crossings</td>
<td>• Independence with the piers in absorbing the impact energy</td>
<td>• Limit energy absorption ability</td>
<td>• High restoration costs</td>
</tr>
<tr>
<td></td>
<td>• River crossings</td>
<td>• Economy in moderate water depths</td>
<td>• Large scale vessel damage</td>
<td>• Low long term maintenance costs</td>
</tr>
<tr>
<td>Pile supported protection systems</td>
<td>• Retrofit of the existing structures</td>
<td>• Moderate energy absorption ability</td>
<td>• Non-efficiency against high energy collisions</td>
<td>• High construction costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Ability to deviate or deflect the errant vessel</td>
<td>• Limitation of use due to deep water</td>
<td>• High restoration costs</td>
</tr>
<tr>
<td>Fender protection systems</td>
<td>• Inland waterways</td>
<td>• Guidance for ship navigation</td>
<td>• No-efficiency in the medium to high energy collision event</td>
<td>• Moderate construction costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Ability to deflect the errant vessel</td>
<td></td>
<td>• High restoration costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Provision of an anti-sparking, rubbing surface</td>
<td></td>
<td>• High long term maintenance costs</td>
</tr>
</tbody>
</table>
2.3.2 New crashworthy protection device

Recently, a new crashworthy protection device against ship-bridge collision is developed to protect both the bridge and ship. It consists of hundreds of the steel-wire-rope coil (SWRC) connected in parallel and series (Wang et al. 2008). The dynamic numerical simulation results indicate that the peak of impact force markedly decreases due to the high compliance (low wave impedance) and viscous energy dissipation characteristic of SWRCs. Particularly, the new device enables the ship having enough time to turn its navigating direction away; consequently, a large percentage of initial kinetic energy of ship is carried by the turning-away ship.

The advantages of this new device are: a dramatic reduction of the impact force, especially at the initial stage of collision; the ability of prolonging the collision duration under low impact force, so that ship can be turned away and the kinetic energy transmitted to bridge decreases; and the ability of absorbing more exchanged energy to reduce the damage to the ship.

2.4 Surveillance and Warning Systems

Through analyzing ship-bridge collision accidents, an important fact has been discovered that a great part of fatalities resulted from the miss driving into the gap by the uninformed motorists who could not see the bridge segments. The reported disasters of the Queen Isabella Memorial Bridge in the USA and Highway 325 Bridge over the Jiujiang River in China are typical examples of this situation. Although the state-of-the-art technique advance may lead to reasonable resistance capacity of bridge components, there still exists the possibility of span collapse of
bridges due to various unpredicted factors. Therefore, attentions are sharply focused on developing detection and warning systems which can detect catastrophic failures of bridge and then warn the bridge users timely. The design principles of the surveillance and warning system are: simplicity, reliability without false positives, and use of off-the-shelf components.

In 1983, an attempt to provide such a system was made to identify alternative systems which could be installed on the Sunshine Skyway Bridge during its reconstruction (FHWA 1983). In this study, the advantages and disadvantages were identified for different system elements including various prevention, detection and warning devices. The alternative systems were then developed by combining these system elements. At last, a quantitative evaluation was conducted for the alternative systems by numerically weighting goal achievements. The alternatives with higher total goal achievement values were considered to be the best choices.

After the collapse of the Queen Isabella Memorial Bridge in 2001, the Texas Department of Transport (DOT) took the possibility of collapse of the spans in the future into consideration when reopening the bridge (Mercier et al. 2005). They installed a detection system on the bridge, which can detect a span collapse and warn motorists to stop. The system consists of a fiber-optic cable that carries a current under the bridge deck for the $2^{1/2}$-mi length of the bridge. A span collapse will break the current. The system will initiate flashing red lights to tell motorists on the bridge to stop, close gates at each end of the bridge to keep additional cars off, and send alarms to DOT and local law enforcement to notify them of the event.
CHAPTER 3
OPTIMAL SENSOR PLACEMENT FOR DAMAGE DETECTION OF BRIDGES SUBJECT TO SHIP COLLISION

3.1 Introduction

Hundreds of ship-bridge collision accidents are recorded around the world every year (Wardhana and Hadipriono 2003; Mercier et al. 2005). Instead of leading to the collapse of whole bridges, most of them threaten the safety of bridges by causing invisible structural damage (Svensson 2006). Post-collision damage and condition assessment is thus of significant importance for decision making on whether closure of the bridge to traffic is necessary and for planning the consequent bridge strengthening or retrofitting. On-line SHM systems have been successfully implemented on bridges worldwide (Ko and Ni 2005), with which the structural responses of an instrumented bridge during any ship collision event can be monitored automatically (Ko et al. 2005; Song et al. 2007). These monitoring data are very useful for post-collision damage detection (Zhou et al. 2006; Yun et al. 2008). The damage information can be extracted by post-processing of the monitoring data. However, the damage information contained in the monitoring data, which is critical for damage detection, is largely dependent on the sensor layout. It is therefore worthwhile to investigate the optimal sensor placement (OSP) problem.

A variety of OSP methods have been proposed in the past two decades, which mainly serve two purposes: modal identification and damage detection. Most of the
approaches are based on the concept of choosing the best subset, which is the optimal or near optimal solution of the objective function, among all the locations of the candidate sensors. The approaches differ only in their choice of objective function and optimization algorithm (Worden and Burrows 2001).

The influential approaches in the context of modal identification include the effective independence (EI), kinetic energy (KE), eigenvalue vector product (EVP), average driving-point residue (ADPR), and effective independence driving-point residue (EI-DPR) methods (Kammer 1991; Heo et al. 1997; Meo and Zumpano 2005; Li et al. 2007; Rao and Anandakumar 2008; Bar thorpe and Worden 2009). In the EI method, the final sensor configuration tends to maximize the trace and determinant of the fisher information matrix (FIM) (Kammer 1991). The sensor location that contributes least to the linear independence of the target modal partitions is deleted at each iteration until the required number of sensors is achieved. However, the sensor set can also be expanded iteratively to include those sensor locations that offer the greatest increase in the determinant of the FIM (Kammer 2005). The KE method assumes that the sensors will provide higher signal-to-noise ratio for mode-shape identification if the sensors are placed at points of maximum KE of a structural system (Heo et al. 1997; Meo and Zumpano 2005). The EI and KE methods produce similar results and the inherent mathematical connection between them has been revealed (Li et al. 2007). The EVP method computes the product of eigenvalue components for candidate sensor locations over a range of modes; a maximum of this product implies a candidate measurement point (Bar thorpe and
Worden 2009). It can prevent the choice of sensors placed on nodal lines of a vibration mode and maximize their vibration energy (Meo and Zumpano 2005). The ADPR method chooses the coordinates which make the highest (weighted) average contribution to the mode-shapes (Barthorpe and Worden 2009). The EI-DPR method multiplies the candidate sensor contribution of EI by the corresponding ADPR coefficient, leading to a greater likelihood of sensors being placed in areas of high signal strength (Meo and Zumpano 2005; Barthorpe and Worden 2009).

Compared with the OSP methods focusing on modal identification, the OSP methods for the purpose of damage detection are far from maturing. Some researchers developed the methods based on eigenvector sensitivity analysis of structural FE models (Cobb and Liebst 1997; Shi et al. 2000). Xia and Hao (2000) proposed a method to select the measurement points by maximizing the sensitivity of a residual vector to structural damage, and minimizing the sensitivity of the damage to measurement noise.

The genetic algorithm (GA) and simulated annealing (SA) have been used in the OSP problem as alternatives to the EI method. Worden and his co-investigators (Worden and Staszewski 2000; Worden and Burrows 2001) combined a neural network approach for locating and classifying faults with the GA and SA, respectively. The neural network was trained using mode-shape curvatures provided by an FE model of a cantilever plate. The inverse of the probabilities of misclassification for different damage conditions obtained from the neural network was employed as the fitness function for the GA and SA. Guo et al. (2004) defined
an objective function from the matrix of sensitivity coefficients of the mode-shape changes with respect to the damage vector. Maximizing the objective function would lead to the best estimate of damage coefficients. They proposed an improved GA method to search for the optimal sensor locations.

Some researchers introduced the concept of information content in the measured responses, and selected the degrees of freedom (DOFs) for sensor placement with intent to maximize the information content (Udwadia 1994; Heredia-Zavoni and Esteva 1998; Datta et al. 2002). Papadimitriou (2005) extended the concept of information content and developed a statistical method for optimally placing sensors for the purpose of parameter identification and damage detection within the Bayesian statistical framework (Beck and Katafygiotis 1998). The optimality criterion adopted for the sensor placement is the information entropy, which is a unique measure of the uncertainties in model parameters.

Souza and Epureanu (2008) proposed a method on the assumption that the damageable regions (hot spots) are a priori known and sensors are placed at the hot spots; if it is not possible or additional sensors are being used, a generalized EI distribution vector method is applied for the remaining sensors.

No investigation has been reported on dealing with the OSP problem for post-collision damage detection of bridges. An important aspect of this specific OSP problem is concerned with the unknown or uncertain ship collision position in the sensor placement design stage, which has not yet been addressed. When applied to large-scale structures, the sensitivity-based and information-entropy-based OSP
methods for the purpose of damage detection also suffer some limitations. They define the damage measurability of a sensor location based on the sensitivity matrix with respect to the model parameters of each element. However, for large-scale structures like cable-stayed and suspension bridges, it is very difficult to do so due to the huge number of structural elements. To investigate the specific OSP problem targeting damage detection of bridges subject to ship collision, there is a need to develop a feasible OSP method that is not only effective for different ship collision positions, but can also break through the limitations of the existing methods when applied to large-scale bridges. In view of the above, the OSP method presented in this study, which is developed based on the concept of information entropy (Papadimitriou 2005), aims to deal with the uncertainty of ship collision position for large-scale bridges.

3.2 Methodology

3.2.1 Information entropy

The information entropy proposed by Papadimitriou and his co-investigators (Papadimitriou et al. 2000; Papadimitriou 2005), which is a direct measure of the uncertainty in model parameter estimates, is adopted herein as the optimality criterion for sensor placement purposed for post-collision damage detection. The concept behind the information-entropy-based method and the theoretical background of the information entropy are briefed below.

When a structure suffers from damage, the measured structural response and the model response prediction satisfy the following equation
\[ y(m) = L_0 q(m; \theta) + n(m; \theta) + n_d(m; \theta) \tag{3.1} \]

where \( y(m) \in \mathbb{R}^{N_s} \) refers to measurement data; \( N_s \) is the number of measured DOFs; \( q(m; \theta) \in \mathbb{R}^{N_s}, m = 1, \ldots, N \) is the sampled response time histories computed at all \( N_d \) candidate sensor placement DOFs from a particular structural model with the model parameter set \( \theta \), and is depended on the loading applied on the structure (in this study, the impulse loading is considered because the ship-bridge collision force is of short-time, high energy and similar to impulse); \( L_0 \in \mathbb{R}^{N_s \times N_d} \) is the observation matrix comprising zeros and ones and maps the candidate DOFs to the measured DOFs; \( n(m; \theta) \in \mathbb{R}^{N_s} \) is the model prediction error due to modeling error and measurement noise; \( n_d(m; \theta) \in \mathbb{R}^{N_s} \) is the change in responses due to damage and regarded as damage information. It is known from Equation (3.1) that the damage information \( n_d(m; \theta) \) is polluted by the model prediction error \( n(m; \theta) \). Therefore, in order to relatively maximize the damage information content in the monitoring data, the model prediction error \( n(m; \theta) \) should be minimized. As the information entropy can be viewed as a measure of model prediction error (Papadimitriou 2005), it is reasonable to minimize the information entropy in the OSP problem.

The information entropy is defined by

\[ H(\delta, D) = -\int p(\theta | D) \ln p(\theta | D) d\theta \tag{3.2} \]

where \( \delta \in \mathbb{R}^{N_d} \) is the sensor configuration vector, with elements \( \delta_i = 1 \) if \( i \) th DOF is measured and \( \delta_i = 0 \) if \( i \) th DOF is not measured; \( \theta \) is the model parameter set; \( p(\theta | D) \) is probability density function updated using the
measurement data $D$. The information entropy depends on the available data $D = D(\delta)$ and the sensor configuration $\delta$.

In the sensor placement design stage, where the measurement data are not available, an asymptotic approximation of the information entropy is used as the optimality criterion for the OSP problem and it is expressed as

$$
H(\delta, D) \sim H(\delta; \theta_0, \sigma_0) = \frac{1}{2} N_\theta \ln(2\pi) - \frac{1}{2} \left[ \det h(\theta_0, \sigma_0; \delta) \right]
$$

(3.3)

where $\theta_0$ is the nominal values of the parameter set $\theta$ chosen by the designer to represent the system, $\sigma_0^2$ is the nominal value of the prediction error $\sigma^2$ (given by $\sigma^2 = \frac{1}{NN_\theta} \sum_{m=1}^{N} \| y(m) - L_0 q(m; \theta) \|^2$) and is chosen by the designer to represent the system; $N_\theta$ is the number of model parameters; and $h(\theta_0, \sigma_0; \delta)$ is an $N_\theta \times N_\theta$ positive definite matrix defined and asymptotically approximated by

$$
h(\theta_0, \sigma_0; \delta) \sim \frac{1}{\sigma_0^2} Q(\delta, \theta_0)
$$

(3.4)

The matrix $Q(\delta, \theta_0)$ appearing in Equation (3.4) is a positive semi-definite matrix of the form

$$
Q(\delta, \theta_0) = \sum_{j=1}^{N_\delta} \delta_j P^{(j)}(\theta_0)
$$

(3.5)

which is known as the fisher information matrix (Udwadia 1994) and contains the information about the values of the parameter set $\theta$ based on the data from all measured locations specified in $\delta$. The matrix $P^{(j)}(\theta_0)$ is a positive semi-definite sensitivity matrix given by

$$
P^{(j)}(\theta_0) = \sum_{m=1}^{N} \nabla_\theta q_j(m; \theta_0) \nabla_\theta^T q_j(m; \theta_0)
$$

(3.6)
in which $\nabla_\theta = \left[ \frac{\partial}{\partial \theta_1}, \ldots, \frac{\partial}{\partial \theta_N} \right]^T$ is the gradient vector with respect to the parameter set $\theta$. The asymptotic value of the information entropy, given in Equation (3.3), does not depend explicitly on the measurement data $D$. The only dependence of the information entropy on the data comes implicitly through the nominal values $\theta_0$ and $\sigma_0^2$.

It should be noted that the first term on the right hand side of Equation (3.3) is constant for a given model, while $\det h(\theta_0, \sigma_0; \delta)$ in the second term, which is calculated from Equations (3.4) to (3.6), can be regarded as a measure of the sensitivity of responses at the measured DOFs with respect to the model parameter set $\theta$. Hence, minimizing the information entropy is in fact maximizing the sensitivity of the measured responses with respect to the model parameter set $\theta$. This does make sense because if the sensitivity of the measured responses is maximized, a sudden change in $\theta$ can be detected more accurately from the changes of responses.

### 3.2.2 Formulation of OSP problem with given ship collision position

The OSP problem is first formulated given a ship collision position. Suppose that $N_o$ out of $N_d$ ($N_o \leq N_d$) DOFs are selected as the optimal sensor configuration by minimizing the information entropy. That is

$$\delta_{opt} = \arg \min_{\delta} H(\delta; \theta_0, \sigma_0)$$

where $\delta$ is the sensor configuration vector comprising $N_o$ ones and $N_d - N_o$ zeros. The sensitivity matrix $P^{(i)}(\theta_0)$ used for calculating the information entropy is obtained using impulse response time histories at all candidate sensor locations.
given a ship collision position. The direction of impulse force is assumed to be along the lateral direction of the bridge. The impulse responses are used because the ship-bridge collision force is of short-time, high energy and similar to impulse. Since an exhaustive search over all sensor configurations for finding the optimal one is prohibitive for a structure with a huge number of DOFs, the $N_o$ DOFs for placing sensors are selected sequentially by placing one sensor at a time in the structure at the DOF that results in the largest reduction in the information entropy. This may lead to a sub-optimal solution, but the resulting sub-optimal solution is very close to the exact optimal solution (Papadimitriou 2004).

### 3.2.3 Formulation of OSP problem with uncertain ship collision position

Because the ship collision position is uncertain in the sensor placement design stage, the ideal sensor configuration should be effective for any possible ship-bridge collision scenario. To achieve this goal, the OSP problem is now formulated as a multi-objective optimization with each objective representing one scenario. The result obtained from the multi-objective optimization is a compromise sensor configuration trading off the qualities of information for all scenarios. In this study, the occurrence probabilities of all possible ship-bridge collision scenarios are assumed to approximately follow the uniform distribution, so all the ship-bridge collision scenarios are considered equally important. Hence, the compromising sensor configuration is designed to provide almost equally informative data for all the scenarios.

The ship-bridge collision scenarios are simulated by applying an impulse force
at different nodes on the bridge deck and towers. Then the acceleration responses at all candidate DOFs for placing sensors under the impulse excitation are obtained by modal superposition or numerical integration method in each scenario. It has been found that the characteristics of structural responses (frequency response functions or FRFs) under the impulse excitation are quite similar in the scenarios whose impact positions are adjacent. For simulating ship-bridge collision scenarios, therefore, it is not necessary to apply the impulse force at every node in possible collision regions. Instead, the possible ship collision regions can be discretized into a small number of nodes to reduce the unnecessary computational effort, given that the scenarios corresponding to the discretized positions are able to capture all kinds of characteristics of ship-collision-induced structural responses.

Let $J_i(\delta)$ be the objective function for the $i$th scenario, the multi-objective problem is formulated to find the optimal $N_o$ DOFs by simultaneously minimizing the following objectives for individual scenarios,

$$y = J(\delta) = (J_1(\delta), J_2(\delta), \ldots, J_{N_J}(\delta))$$ \quad (3.8)

where $J_i(\delta)$ is the information index defined to evaluate the effectiveness of a sensor configuration $\delta$ for the $i$th scenario and represents the normalization required for accurate combination of multiple objective functions. It is expressed as

$$J_i(\delta) = \frac{H(\delta) - H(\delta_{\text{best},i})}{H(\delta_{\text{worst},i}) - H(\delta_{\text{best},i})} \quad (i = 1, 2, \ldots, N_J)$$ \quad (3.9)

where $N_J$ is the number of scenarios; the notation $H(\delta) \equiv H(\delta; \theta_0, \sigma_0)$ is introduced for convenience, while $H(\delta_{\text{best},i})$ and $H(\delta_{\text{worst},i})$ are the information entropies corresponding to the optimal and worst sensor configurations $\delta_{\text{best},i}$ and
The optimal result is achieved by first finding the Pareto optimal solutions, which are a set of solutions that cannot be improved in any objective without causing degradation in at least one other objective (Papadimitriou 2005). All the Pareto solutions are more informative for some scenarios at the expense of being less informative for others except for one, which provides almost equally informative data for all the scenarios. This special Pareto solution will be chosen as the optimal solution. In order to find it, a bound value $J_0 (J_0 = \max \{ J_i \})$ is introduced for individual Pareto solution, so that the values of all the objectives $J_i$ are not more than $J_0$. The Pareto solution whose bound value $J_0$ is the minimum gives no preference to any scenario by providing almost equally informative data for all the scenarios and is the optimal solution.

### 3.2.4 Parameterized damage monitoring model

In order to detect sudden changes, which are induced during ship collision accidents that usually last for a few minutes, in local stiffness, the response data measured from the optimal sensor layout should be sensitive to the local damage in each element. It is therefore desired to calculate the sensitivity matrix with respect to the stiffness parameters of each element and minimize the corresponding information entropy in the OSP. For small-scale structures whose number of elements is not very large, it is possible to conduct the OSP in such a way. For large-scale structures, however, including the stiffness parameters of each element in the model parameter set will make the computation of the sensitivity matrix prohibitive due to the huge
number of elements.

To overcome this problem, a parameterized damage monitoring model (DMM) which can be used to describe the input-output behavior of a bridge is introduced to reduce the number of model parameters. The DMM has a significantly reduced number of parameters through a two-step parameterization. In the first step, critical elements that are in need to be monitored during ship-bridge collision accidents are determined by criticality analysis and engineering judgment. Then the stiffness parameters (e.g., the Young’s modulus) of these elements are selected. Because the number of critical elements will still be vast for a large complex structure, considering the Young’s moduli of all the critical elements to be the parameters of the DMM will cause significant computational amount. Therefore, in the second step, to further reduce the computational effort, the critical elements are grouped according to their positions, namely, the adjacent elements are classified into the same group. By assuming the Young’s moduli of all the elements in the same group to be the same and fully correlated, the Young’s moduli of the elements in a group can be represented by one parameter. However, according to the engineering judgment, in the circumstance where the diversity of Young’s moduli of different elements in one group is so significant that the above assumption does not hold, the elements in that group should be divided into several subgroups, in order to ensure the Young’s moduli of different elements in each subgroups are the same or in close proximity and change at the same rate. Then the Young’s moduli of the elements in each subgroup will be selected as a DMM parameter.
3.2.5 Identification of sensitive regions and determination of sensor configuration

It has been shown from numerical results that the selected $N_o$ DOFs for sensors placement by the multi-objective optimization algorithm tend to aggregate in some regions for a long-span bridge. This is because the responses measured in some regions are much more sensitive to the change in DMM parameters than those measured in other regions; always more than one DOF in each of these regions would be preferentially selected. The regions where more than one DOF is selected are termed sensitive regions. In order to identify the sensitive regions, the average linkage method (Morse 1980), which is a clustering analysis method, is utilized to classify the selected DOFs into several clusters (say, $N_c$, $2 \leq N_c \leq N_o$). Then the clusters with more than one selected DOF are identified as the sensitive regions. On one hand, the information contained in the different selected DOFs in a sensitive region would be similar, and hence there is no need to place sensors at all the selected DOFs in each sensitive region. On the other hand, however, as the sensitive regions are dominant over other regions in terms of the amount of damage information, these regions should have a certain redundancy to guarantee that in case of malfunction of some sensors in these regions, it is still possible for the entire monitoring system to acquire sufficient damage information. In view of keeping a certain redundancy, a few selected DOFs (say, $s_i$, $2 \leq s_i \leq n_i$, $i=1,\ldots,N_c$, $n_i$ is the number of selected DOFs in a sensitive region) in each sensitive region are randomly chosen for placing sensors. Other selected DOFs in each sensitive region
do not need to be placed with sensors. Therefore, the final optimal sensor configuration consists of $s_i$ selected DOFs in each sensitive region and all the selected DOFs in non-sensitive regions.

3.2.6 **Procedure of implementation**

The computational procedures for the proposed OSP method are as follows:

Step 1: Discretize ship collision positions and define representative ship collision scenarios.

Step 2: Formulate DMM.

Step 3: According to Equation (3.6), calculate the sensitivity matrix $P^{(j)}(\theta_\phi)$ with respect to the parameters of the formulated DMM, using the impulse response time histories at all candidate DOFs for each of the defined representative ship collision scenarios.

Step 4: Find the Pareto optimal solutions of $N_o$ DOFs for all representative ship collision scenarios by iteratively expanding the Pareto optimal solutions of $n-1$ DOFs to Pareto optimal sensors of $n$ DOFs ($n=1,2,\ldots,N_o$). In each iteration, the following sub-steps are implemented.

1. Find the optimal and worst configurations $\delta_{\text{best},i}$ and $\delta_{\text{worst},i}$ of $n$ DOFs for each scenario by sequentially placing one sensor at a time in the structure at the DOF that results in the largest reduction in the information entropy, which is calculated from sensitivity matrix by use of Equations (3.3) to (3.5); and get the corresponding $H(\delta_{\text{best},i})$ and $H(\delta_{\text{worst},i})$ for each scenario.

2. Find all the possible sensor configurations of $n$ DOFs by adding one DOF...
to the Pareto optimal solutions of \( n - 1 \) DOFs obtained in last iteration.

(3) Calculate the information indices \( J_i (t = 1,2,\ldots,N_f) \) for each possible sensor configuration according to Equation (3.9).

(4) Compare the information indices among all possible sensor configurations.

If there exist two sensor configurations \( a \) and \( b \) which satisfy the condition \( J_i^a \leq J_i^b, \text{for } i = 1,2,\ldots,N_f \), the sensor configuration \( b \) will be deleted. After deleting all such sensor configurations, the remaining possible sensor configurations are the Pareto optimal solutions.

Step 5: Select the Pareto optimal solution with the minimum bound value \( J_o \) as the optimal sensor configuration.

Step 6: Classify the optimally selected \( N_o \) DOFs into \( N_c \) clusters by use of the average linkage method and identify the sensitive region.

Step 7: Determine the final optimal sensor configuration by keeping \( s_i \) selected DOFs in each sensitive region and all the selected DOFs in non-sensitive regions.

3.3 Case Study

3.3.1 Introduction of TKB

The cable-stayed TKB in Hong Kong is considered to demonstrate the feasibility and effectiveness of the proposed method. Considering that the time histories of the measured data during different ship-bridge collision scenarios are not available yet and it may take long observation period to get such data and it is difficult to carry out field experiment for large scale bridges, the numerical
simulation is conducted in this study to investigate the different ship-bridge collision cases. As shown in Figure 3.1, the TKB is a three-tower cable-stayed bridge with two main spans of 448 m and 475 m respectively, and two side spans of 127 m each (Ni et al. 2005). The bridge deck is separated into two carriageways with a width of 18.8 m each, between them being slender single-leg towers. Each carriageway consists of two longitudinal steel girders (outer and inner girders) along the deck edges with steel crossgirders at 4.5 m intervals, and a concrete slab on top. The two carriageways, with a 5.2 m gap, are linked at 13.5 m intervals by connecting crossgirders. The bridge deck is supported in the transverse direction at the three towers, and on the northern end pier and the southern abutment. In the longitudinal direction, the deck is supported only at the central tower. On both ends of the bridge, the deck is vertically connected by rocker bearings into the northern Ting Kau end pier and the southern Tsing Yi abutment. The bridge has three single-leg towers supporting two main spans and two side spans. Each concrete tower is composed of three cross sections at different heights. There are 384 main stay cables (excluding stabilizing cables) in four planes anchored to the deck edge girders at 13.5 m
intervals. Because of the slender towers, eight longitudinal stabilizing cables with lengths up to 465 m are used to diagonally connect the top of the central tower to the deck near the side towers. In addition, both the central and side towers are stiffened by a total of 64 transverse stabilizing cables in the lateral direction.

A precise three-dimensional FE model of the TKB, as illustrated in Figure 3.2, has been developed by use of the commercial software package ABAQUS® for the purpose of parameter sensitivity analysis and ship collision simulation. The cables and girders of the bridge are simulated by use of beam element B31, the deck is simulated using shell element S4R, and the bearings are simulated using truss element T3D2. The global coordinate system adopted to represent the model is also shown in Figure 3.2, where the x-axis is allocated along the bridge longitudinal direction, the y-axis is along the vertical direction, and the z-axis is in the lateral direction.

3.3.2 Discretization of ship collision positions and formulation of DMMs

With the formulated FE model of the TKB, the acceleration responses at all
Table 3.1 Natural frequencies and description of mode shapes for the first 120 modes

<table>
<thead>
<tr>
<th>Mode order</th>
<th>Frequency (Hz)</th>
<th>Global* or local** mode</th>
<th>Description of mode shape</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1660</td>
<td>Global</td>
<td>Predominantly vertical bending</td>
</tr>
<tr>
<td>2</td>
<td>0.1761</td>
<td>Global</td>
<td>Predominantly lateral bending</td>
</tr>
<tr>
<td>3</td>
<td>0.1873</td>
<td>Global</td>
<td>Predominantly lateral bending</td>
</tr>
<tr>
<td>4</td>
<td>0.2131</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>5</td>
<td>0.2138</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>6</td>
<td>0.2268</td>
<td>Global</td>
<td>Predominantly lateral bending</td>
</tr>
<tr>
<td>7</td>
<td>0.2277</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>8</td>
<td>0.2288</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>9</td>
<td>0.2302</td>
<td>Local</td>
<td>Cable mode+ tower bending</td>
</tr>
<tr>
<td>10</td>
<td>0.2321</td>
<td>Local</td>
<td>Cable mode+ tower bending</td>
</tr>
<tr>
<td>11</td>
<td>0.2402</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>12</td>
<td>0.2421</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>13</td>
<td>0.2501</td>
<td>Global</td>
<td>Predominantly torsional bending</td>
</tr>
<tr>
<td>14</td>
<td>0.2602</td>
<td>Global</td>
<td>Coupled torsional and lateral</td>
</tr>
<tr>
<td>15</td>
<td>0.2917</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>16</td>
<td>0.2925</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>17</td>
<td>0.2934</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>18</td>
<td>0.2943</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>19</td>
<td>0.3045</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>20</td>
<td>0.3050</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>21</td>
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<td>Local</td>
<td>Cable mode</td>
</tr>
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<td>Local</td>
<td>Cable mode</td>
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<td>0.3298</td>
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<td>Predominantly vertical bending</td>
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</tr>
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<td>26</td>
<td>0.4232</td>
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</tr>
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<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
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<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>31</td>
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<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>32</td>
<td>0.4542</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>33</td>
<td>0.4543</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
<td>34</td>
<td>0.4545</td>
<td>Local</td>
<td>Cable mode</td>
</tr>
<tr>
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<td>120</td>
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<td>Predominantly vertical bending</td>
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Remark: *Global mode involving the dynamics of the bridge deck and piers  
**Local mode involving the dynamics of cables and partial deck
candidate DOFs for placing sensors under the impulse excitation are evaluated for each scenario. In this study, the modal superposition method taking into account the first 120 modes is applied to calculate the impulse responses. The natural frequencies and description of mode shapes for the 120 modes are detailed in Table 3.1. There are 48 modes involving the dynamics of the bridge deck and piers, and 72 modes only involving the dynamics of cables and partial deck. The modal damping is assumed to be 0.02, and the impulse responses of 30 seconds are included in the following computation. By comparing the FRFs obtained under a wide spectrum of excitation points, it is found that all kinds of characteristics of the ship-collision-induced structural responses can be generated by applying the impulse excitation on 11 selected nodes. These 11 nodes are selected as representative ship collision positions (scenarios) as shown in Figure 3.3. The direction of impulse excitation in each scenario is along the lateral direction of the bridge, as indicated in Figure 3.3. Because the bridge is symmetric with respect to the longitudinal central line, the collision positions are considered only at one side of the central line.

![Figure 3.3 Ship collision positions in different scenarios](image)
A criticality and vulnerability analysis has been conducted on the TKB with the purpose of designing a SHM system for this bridge (Flint and Neil Partnership 1998; Wong 2004). The critical elements in need to be monitored during ship-bridge collision accidents are thus determined. Then the critical elements are grouped according to their positions. The critical elements, whose locations are adjacent and Young’s moduli are the same, are classified into the same group. Figures 3.4 and 3.5 show two DMMs for the TKB, where DMM1 is obtained by considering the critical elements in both bridge deck and towers with the critical elements being classified into five groups, and DMM2 is obtained by considering only the critical elements in

![Figure 3.4 Damage monitoring model 1 (DMM1) for TKB](image)

![Figure 3.5 Damage monitoring model 2 (DMM2) for TKB](image)
bridge deck with the critical elements being classified into six groups. The Young’s modulus of the elements in one group is selected as a DMM parameter $\theta_i$.

3.3.3 **OSP for post-collision damage detection**

The proposed OSP method is applied to find the optimal sensor configuration for post-collision damage detection of the TKB in respect of the two DMMs shown in Figures 3.4 and 3.5. While the exploration on DMM1 is detailed to demonstrate the feasibility and effectiveness of the proposed method, only the OSP results are presented for DMM2 without detailed discussions. Firstly, the sensitivity matrix $P^{(j)}(\theta_0)$ with respect to the parameters of DMM1 is calculated using the impulse response time histories at all the 864 candidate DOFs ($N_d = 864$, including the longitudinal, transverse and vertical DOFs of 288 candidate sensor locations in the deck) for each of the 11 representative ship collision scenarios. The information entropy is then obtained from the sensitivity matrix. Next, 60 DOFs ($N_o = 60$) for all the 11 collision scenarios are selected by the multi-objective optimization algorithm that simultaneously minimizes the information index $J_i$ for each scenario. The result is presented in Figure 3.6, where all the 60 DOFs selected are

![Figure 3.6 Clusters of sensor locations for DMM1](image)

| Indicates sensor locations |
| Indicates the clusters of sensor locations |

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along the vertical direction. The result indicates that the vertical acceleration responses are more sensitive to the parameters of DMM1 than the longitudinal and transverse acceleration responses. Another observation is that all the sensor locations selected are at the two sides of the deck rather than the center of the deck. This implies that the acceleration responses at the two sides are more sensitive to the parameters of DMM1 than those at the center of the deck. The average linkage method is used to classify the selected 60 DOFs into 20 clusters ($N_c = 20$, ten for each side of the bridge deck) as shown in Figure 3.6. The clusters with more than one selected DOF are identified as sensitive regions. Finally, the sensor configuration is determined by randomly keeping two ($s_i = 2$, for $i = 1, \ldots, N_c$) selected DOFs in each sensitive region and including all the selected DOFs in non-sensitive regions, as illustrated in Figure 3.7. The clusters of sensor locations and the final optimal sensor configuration for DMM2 are shown in Figures 3.8 and 3.9, respectively. Again, all the optimally selected DOFs are along the vertical direction.

![Indicates sensor locations](image)

Figure 3.7 Final optimal sensor configuration for DMM1

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3.3.4 Verification of optimal sensor configuration

In order to verify the effectiveness of the obtained optimal sensor configuration for DMM1 under different ship-bridge collision scenarios, 11 damage cases are studied. Without loss of generality, only the results of the first two cases are detailed, while the results of the other cases, which are able to represent the typical cases where ship-bridge collision positions are at the piers and deck respectively, are briefly summarized at the end of this section. In Case 1, the 6th ship-bridge collision scenario, in which the collision position is at the bottom of the central tower as shown in Figure 3.3, is assumed. The damage is simulated as a 15% reduction of the
parameter $\theta_3$ as shown in Figure 3.4. In Case 2, the 3rd ship-bridge collision scenario, in which the ship impacts the bridge deck near the center of the left main span as shown in Figure 3.3, is assumed. The damage is simulated as a 15% reduction of the parameter $\theta_2$ as shown in Figure 3.4.

The verification is first conducted for Case 1. The criterion of effectiveness should be related to the amount of damage information provided by the optimally selected DOFs. The change in impulse responses before and after damage can be regarded as the damage information. As a result, the optimally selected DOFs for sensor placement are identified to be effective if the amounts of response change at the optimally selected DOFs are larger than those at the other DOFs.

Figure 3.10 shows a comparison between the response changes at one of the optimal DOFs (the vertical DOF of Node No. 5030) and at one of the non-optimal
DOFs (the longitudinal DOF of Node No. 5001). It is found that with the same
damage extent, the response change at the optimal DOF is much larger than that at
the non-optimal DOF. This indicates that the damage information provided by the
optimal DOF is much more than that provided by the non-optimal DOF. To examine
whether all the optimally selected DOFs are effective in Case 1, a comparison
between the amounts of damage information at all the optimal DOFs and those at the
other DOFs is carried out. The amount of damage information is measured by the
average power of the change in impulse acceleration response, i.e.,
\[ P = \frac{1}{n_t} \sum_{i=1}^{n_t} C(t_i)^2 \]  \hspace{1cm} (3.10)
where \( n_t \) is the number of time points; \( C(t_i) \) represents the change of impulse
acceleration response at the time point \( t_i \), and is given by
\[ C(t_i) = R_d(t_i) - R_u(t_i) \]  \hspace{1cm} (3.11)
where \( R_u(t_i) \) and \( R_d(t_i) \) denote the impulse acceleration responses at the time
point \( t_i \) before and after damage, respectively.

Comparisons of the average power of the change in impulse response between
all the optimally selected vertical DOFs and the other vertical, longitudinal and
transverse DOFs at the left side (as indicated in Figure 3.2) of the bridge deck are
shown in Figures 3.11 to 3.13, respectively. The abscissa indicates the node number
of bridge girders while the ordinate gives the \( P \) value. It is observed in Figure 3.12
that the amounts of damage information at the longitudinal DOFs are negligible in
comparison with those at the optimally selected vertical DOFs. The average value of
\( P \) for all the optimally selected vertical DOFs is \( 2.32 \times 10^{-14} \text{ (m} / \text{s}^2)^2 \), while that
Figure 3.11 (a) Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and other vertical DOFs at left side of bridge deck; (b) Local sensitive regions in TKB model (Case 1)

Figure 3.12 Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and longitudinal DOFs at left side of bridge deck (Case 1)
for all the longitudinal DOFs is $2.41 \times 10^{-16} \text{(m/s}^2)^2$. The average amount of damage information provided by the optimally selected vertical DOFs is 437 times more than that provided by the longitudinal DOFs. It is seen in Figure 3.13 that the amounts of damage information at the transverse DOFs are much less than those at the optimally selected vertical DOFs. The average amount of damage information provided by the optimally selected vertical DOFs is about 15 times compared with that provided by the transverse DOFs, with the average values of $P$ being $2.32 \times 10^{-14} \text{(m/s}^2)^2$ for the former and $6.02 \times 10^{-16} \text{(m/s}^2)^2$ for the latter. This explains why all the selected DOFs are in the vertical direction. From the above observations, the optimally selected DOFs by the proposed method are effective in the sense that they can provide more damage information than most of the other DOFs including all the longitudinal and transverse DOFs.

In Figure 3.11, some regions exhibit ‘peak’ values in the average power of the
response change; they are the regions labeled LSR 1, LSR 2, LSR 3, LSR 4, LSR 5, LSR 6, LSR 7 and LSR 8 in Figure 3.11. These ‘peak’ values indicate that the amounts of the change in impulse responses due to the damage are relatively large in the above regions. These regions are identified as the local sensitive regions for a certain case. The local sensitive regions for this case, where the collision position is at the bottom of the central tower, are distributed at about one third of the two side spans from the abutments, one fourth and three fourths of the two main spans, and the center of the right main span, as shown in Figure 3.11(b). However, the center of left main span does not manifest itself as a local sensitive region, since the TKB is not strictly symmetric. For six out of the total eight local sensitive regions (except for the regions LSR 5 and LSR 6), there is at least one DOF which was selected for sensor placement in each local sensitive region, indicating that the selected DOFs are effective in the sense that they can capture the damage information from most of the local sensitive regions. Based on the above observation, an index $E$ which evaluates the effectiveness of the optimally selected DOFs at one side of the bridge deck in a certain case is defined as

$$E = \frac{N_1}{N_2} \times 100\%$$  (3.12)

where, $N_1$ is the number of the local sensitive regions with at least one optimally selected DOF at one side of the bridge deck, $N_2$ is the total number of the local sensitive regions at the same side of the bridge deck. For the left side of the deck in Case 1, the effectiveness index $E$ is 75%.

Figures 3.14 to 3.16 compare the average power of the change in impulse
Figure 3.14 (a) Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and other vertical DOFs at right side of bridge deck; (b) Local sensitive regions in TKB model (Case 1)
response between all the optimally selected vertical DOFs and the other vertical, longitudinal and transverse DOFs at the right side (as indicated in Figure 3.2) of the bridge deck. In Figure 3.14, seven of the eight identified local sensitive regions have at least one DOF selected for sensor placement, resulting in a higher $E$ value.
87.5% compared with that of the left side of the bridge deck. The comparison between Figures 3.11 and 3.14 indicates that the local sensitive regions for Case 1 are symmetrically distributed at the two sides of the bridge deck. In Figures 3.15 and 3.16, again, the average amounts of damage information at the optimally selected vertical DOFs are much more than those at the longitudinal and transverse DOFs by 440 and 14 times respectively. It is therefore concluded that the optimally selected vertical DOFs at the right side of the bridge deck are also effective for Case 1.

Figure 3.17 (a) Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and other vertical DOFs at left side of bridge deck; (b) Local sensitive regions in TKB model (Case 2)
Figure 3.18 Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and longitudinal DOFs at left side of bridge deck (Case 2)

Figure 3.19 Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and transverse DOFs at left side of bridge deck (Case 2)
Figure 3.20 (a) Comparison of average power of change in impulse acceleration response between optimally selected vertical DOFs and other vertical DOFs at right side of bridge deck; (b) Local sensitive regions in TKB model (Case 2)
The verification results for effectiveness of the optimally selected DOFs in Case 2 are given in Figures 3.17 to 3.22. In Figures 3.17 and 3.20, the local sensitive regions are symmetrically located at about one third of the two side spans from the abutments, one fifth, two fifths, three fifths, and three fourths of the left main span,
as well as one fourth and three fourths of the right main span. It is seen from the comparisons between Figures 3.11 and 3.17, Figures 3.14 and 3.20, that the locations at about one third of the two side spans from the abutments and one fourth and three fourths of the right main span are local sensitive regions in both the two cases. Similar to Case 1, it is found from Figures 3.17 and 3.20 that the optimally selected DOFs measure the damage information from most of the local sensitive regions, with $E = 100\%$ for the left side of the deck and $E = 87.5\%$ for the right side of the deck. Again, the amounts of damage information at the longitudinal DOFs are negligible in comparison with those at the optimally selected vertical DOFs, as shown in Figures 3.18 and 3.21; the amounts of damage information at the optimally selected vertical DOFs are much more than those at the transverse DOFs, as shown in Figures 3.19 and 3.22. This indicates that the optimally selected vertical DOFs are informative in Case 2.

The further numerical investigation indicates that, in the other nine cases, the amounts of damage information at the optimally selected vertical DOFs are much more than those at the longitudinal and transverse DOFs, and the optimally selected vertical DOFs are able to measure the damage information from most of the local sensitive regions, similar to the first two cases. The effectiveness indices for all the damage cases studied are summarized in Table 3.2. It is concluded from the table that the optimally selected DOFs are effective under every ship-bridge collision scenario with a minimum effectiveness index $E = 75\%$. 
Table 3.2 Effectiveness indices of optimally selected DOFs in each damage case

<table>
<thead>
<tr>
<th>Case</th>
<th>Collision scenario</th>
<th>Collision position</th>
<th>Damage position</th>
<th>Damage extent</th>
<th>Side of deck</th>
<th>$N_1$</th>
<th>$N_2$</th>
<th>$E$</th>
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<td></td>
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<td></td>
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<td>8</td>
<td>87.5%</td>
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<tr>
<td>2</td>
<td>3</td>
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</tr>
<tr>
<td></td>
<td></td>
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<td></td>
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<td>$\theta_4$</td>
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<td>7</td>
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<tr>
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<td>10</td>
<td>Node No. 5380</td>
<td>$\theta_4$</td>
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<td>8</td>
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<td>77.8%</td>
</tr>
<tr>
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<td>Node No. 303</td>
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<td>6</td>
<td>83.3%</td>
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<td>7</td>
<td>85.7%</td>
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</tbody>
</table>
3.4 Summary

An information-entropy-based OSP method is proposed to deal with the specific OSP problem for damage detection of bridges subject to ship collision. The DMM accommodating the critical elements is defined by parameterizing the Young’s moduli of the critical elements determined by criticality analysis and engineering judgment. Different ship-bridge collision scenarios are simulated by applying an impulse force at possible collision positions. The impulse acceleration responses at all candidate DOFs are used to calculate the sensitivity matrix with respect to the DMM parameters. Then the information entropy is obtained from the sensitivity matrix to extract the content of damage information inherent in the impulse response time histories. Finally, the OSP for all possible ship collision scenarios is formulated by a multi-objective optimization algorithm that simultaneously minimizes the information entropy index for each scenario. The numerical results have demonstrated the feasibility and effectiveness of the proposed method.

The advantages of the proposed method are as follows: (i) It can handle the uncertainty of ship collision position by formulating the OSP problem as a multi-objective optimization. The final optimal solution provides almost equal information for all the ship collision scenarios without giving any preference to a specific scenario. The numerical example demonstrates that the resulting optimal sensor configuration is effective under different ship collision scenarios; (ii) The sensitive regions with respect to the change in DMM parameters can be identified by clustering analysis; (iii) With the identified sensitive regions, the sensor
configuration can be finally determined by randomly reserving a few selected DOFs in each sensitive region and keeping all the other selected DOFs in non-sensitive regions. It guarantees a redundancy of sensors deployed at the most informative regions, and thus sufficient information can still be obtained even in case some of the sensors are malfunctioned; (iv) The proposed method uses the acceleration responses obtained from the FE analysis, and is therefore applicable in practice to determine the sensor placement prior to field testing; and (v) It leaves some freedom to determine the critical elements of a bridge which are in need to be monitored. This makes the method adaptable in different applications.
CHAPTER 4
SHIP-BRIDGE COLLISION ACCIDENT ALARMING
AND SHIP-COLLIDED BRIDGE CONDITION
ASSESSMENT USING HHT BASED APPROACHES

4.1 Introduction

The bridges over waterways are exposed to the ship-bridge collision risk. The risk of ship-bridge collision is increasing due to the increase in the frequency of commercial ship trips in modern times. There are hundreds of ship-bridge collision accidents worldwide every year. Rather than causing the collapse of whole bridges, most of the accidents may lead to invisible structural damage, which is well hidden behind an apparent structural integrity condition and can bring about hidden danger threatening the bridge safety. As a result, post-collision condition assessment for ship-collided bridges is of great importance when deciding whether to close a bridge to traffic and when planning consequent bridge retrofitting. However, small ship-bridge collision accidents may not necessarily be observed and recorded by the bridge management authority. Therefore, in order to assess the post-collision bridge condition, one should detect the ship collision accident occurrence time from the monitoring data and then alarm the accident at first. Hence, a set of methods that can detect ship-bridge collision accidents and then assess the post-collision bridge condition is in need to be developed.

Thanks to the development of on-line SHM systems which have been successfully implemented on bridges worldwide (Ko and Ni 2005), the structural
responses of the instrumented bridge during any ship collision event can be monitored automatically (Ko et al. 2005). Such monitoring data are very useful for ship-bridge collision accident detection, as well as post-collision bridge condition assessment (Zhou et al. 2006). Signal analysis and processing techniques are essential for bridging the gap between the monitoring data and the ship-bridge collision accident detection and structural condition assessment. In the field of signal processing, the FFT has dominated for decades because of its prowess and simplicity. It has been used to detect changes in signal properties by extracting modal properties or damage-sensitive features from the resulting spectra. However, the FFT, which decomposes a signal by a linear combination of projections onto an infinite-duration trigonometric basis, describes the average characteristics of the signal over the whole time history and is thus unable to capture local features of the signal. Examination of the averaged time-invariant modal properties of a structure neither can typically provide information about whether a system is nonlinear, nor can it identify a sudden change in system properties (Staszewski and Robertson 2007). As a result, the FFT is unsuitable to handle non-stationary and non-linear signals.

Because the ship-collision-induced bridge dynamic responses are transient, non-stationary and possibly non-linear, time-frequency transforms are a good alternative to analyze such signals. Among a spectrum of time-frequency analysis techniques, the wavelet transform and Hilbert-Huang transform (HHT) have received much attention and have been widely used in the SHM field (Kijewski and Kareem 2003; Neild et al. 2003; Yang et al. 2003a; Yang et al. 2003b; Yan et al.
The wavelet analysis could provide local features in both time and frequency domains and has the characteristics of multi-scale and “mathematical microscope” (Cheng et al. 2008). Several researchers have applied wavelet approaches to detect the fault in gear systems (Staszewski and Tomlinson 1994; Wang and McFadden 1995). Hera and Hou (2004) investigated the application of wavelet approach for an American Society of Civil Engineers (ASCE) SHM benchmark problem. They detected the structural damage due to sudden breakage of structural elements and the time when it happened. Wavelet based methods have also been used to identify linear time varying or non-linear systems (Staszewski 1997; Katida 1998; Staszewski 1998; Ghanem and Romeo 2000; Piombo et al. 2000; Lardies and Gouttebroze 2002; Basu et al. 2008). However, the wavelet transform is essentially an adjustable window Fourier transform (Cheng et al. 2008). It also suffers many shortcomings of the FFT analysis: It can only give a physically meaningful interpretation to linear phenomena; it can resolve the inter-wave frequency modulation provided the frequency variation is gradual, but it cannot resolve the intra-wave frequency modulation (Huang et al. 1998; Huang et al. 1999). Another inevitable deficiency of wavelet transform is the energy leakage problem generated by the limited length of the basic wavelet function (Huang et al. 1998; Peng et al. 2005). In addition, the wavelet analysis is not a self-adaptive signal processing method in nature (Cheng et al. 2008). Once the basic wavelet is selected, one will have to use it to analyze all the data (Huang et al. 1998). However, the pre-selected
basic wavelet function is not necessarily appropriate for analyzing all the data. Part of the data which have local oscillations not matching the pre-selected wavelet shape cannot be represented well by the wavelet transform.

The HHT, which is composed of the EMD and Hilbert transform, is adaptive and eliminates the need for spurious harmonics to represent non-linear and non-stationary signals. It has been advocated by illustrating its superior performance in comparison with the wavelet transform in numerous examples (Huang et al. 1998; Kijewski-Correa and Kareem 2006). Yang et al. (2003a; 2003b) proposed a system identification method based on the HHT analysis to identify modal parameters of multi-degree-of-freedom linear systems using measured free vibration data. They also developed HHT based damage detection methods to detect the damage time instants and damage locations (Yang et al. 2004). Varadarajan and Nagarajaiah (2004) utilized the HHT to identify the dominant frequency, which is used in frequency tuning of a novel semiactive variable stiffness-tuned mass damper for response control of a wind-excited tall building.

In spite of the merits of the HHT in analyzing the non-linear and non-stationary signals, it also has some annoying problems. A major drawback of the HHT is the mode mixing during the EMD process. Although the EMD is claimed to have the ability of decomposing the non-linear and non-stationary signals into a set of monotonic components whose instantaneous frequencies can be readily extracted by using Hilbert transform, it has been proved later that the mode mixing problem appears frequently during the EMD process and especially the EMD cannot
decompose narrowband multi-component signals, with the resultant intrinsic mode functions (IMFs) often containing multiple components (Yang 2008). Consequently, the instantaneous frequencies extracted from such IMFs may be irregular and difficult to be interpreted. Also, the EMD would generate undesirable IMFs at the low-frequency region (Peng et al. 2005). Wu and Huang (2009) proposed a new ensemble empirical mode decomposition (EEMD) technique, which sifts an ensemble of white noise-added signal and treats the mean as the final true result, to alleviate the mode mixing problem in the EMD method. Based on their method, Wu and Chung (2009) developed a hybrid method of the EEMD and pure EMD to decompose the vibration signals of rotating machinery, so that the fault of the misaligned shaft can be diagnosed in the Hilbert spectrum as well as the marginal Hilbert spectrum.

In considering the superiority of HHT in representing non-linear and non-stationary signals without any artefacts imposed by the non-locally adaptive limitations of the FFT and wavelet processing, a set of HHT based techniques is developed in the present study for ship-bridge collision accident detection and ship-collided bridge condition assessment with use of monitoring data acquired from an on-line SHM system. The HHT method is firstly employed to develop a viable ship-bridge collision accident alarming method. Subsequently it is adopted to detect the structural damage incurred during ship collision. However, in order to overcome the aforementioned deficiencies of HHT, a hybrid HHT based method that combines the EMD, FFT, band-pass filter and Hilbert transform is proposed to interpret the
condition of ship-collided bridges. The instantaneous frequencies and transient energy distributions of ship-collision-induced bridge responses obtained by Hilbert spectrum are analyzed in the time-frequency domain, and are compared with the traditional power spectral densities obtained by the FFT and those by the wavelet transform. The measured acceleration responses of the suspension Jiangyin Bridge during a ship collision event are utilized to testify the effectiveness of the HHT based techniques in alarming the ship-bridge collision accident and assessing the condition of ship-collided bridges.

4.2 Ship-bridge Collision Accident Alarming

At about 20:14 on June 2, 2005, the suspension Jiangyin Bridge in Jiangsu, China, experienced a ship collision accident. A heavy pile-driving boat impacted the bridge deck when it navigated through the route. The structural responses during the accident were measured by the on-line SHM system permanently installed on the bridge. The monitoring data acquired by the accelerometers installed on the bridge will be analyzed with the intention of formulating a viable accident alarming method.

4.2.1 Preliminary analysis of the monitoring data and motivation of the proposed method

There are five lateral and ten vertical accelerometers installed on the central line and two sides of the deck of the Jiangyin Bridge, respectively, as shown in Figure 4.1. The measured lateral and vertical acceleration responses during the ship collision within one-hour duration are shown in Figures 4.2 and 4.3, respectively.
Figure 4.1 Deployment of accelerometers on deck of Jiangyin Bridge

Figure 4.2 Measured lateral acceleration responses during ship collision: (a) AD5CL; (b) AD7CL; (c) AD9CL; (d) AD11CL; and (e) AD13CL

The sampling frequency is 50 Hz. Since the measured vertical acceleration signals from the sensor AD11EV are obviously abnormal (see Figure 4.3(g)), they are excluded in the analysis. The abnormal data appear about three minutes later than the ship collision accident, so this may have some connection with the accident. It is
observed from Figures 4.2 and 4.3 that although the ship collision accident can be detected by observing a sudden increase in the amplitudes of raw data of the bridge responses in the lateral direction, the changes of vibration amplitudes in the vertical direction are not clear when observing raw data sequences in the presence of environmental uncertainty, ambient vibration and measurement noise. It is not surprising because the ship collision force is usually along the lateral direction and leads to large energy input in the lateral direction. However, after carefully examining the data, it is found from Figure 4.2 that the data from three out of the total five lateral accelerometers (see Figures 4.2(a), (b), (d)) show several peaks and spikes especially after the ship collision accident occurred. This phenomenon may be due to heavy trucks passing through or the emergency action taken in the accident.
If the exact time of the accident is completely unknown, these alias peaks and spikes may induce a false alarm. Hence, just using the lateral responses to alarm the accident is not reliable enough given the existence of ambient effects. It is found from Figure 4.3 that the vertical accelerometers are relatively less corrupted by ambient effects. Among the data measured from the nine vertical accelerometers (the sensor AD11EV has been excluded), only two sets of data (see Figures 4.3(a), (h)) show several peaks and spikes. Hence, the vertical responses should also be utilized in the accident alarming. Conceptually, if accident happening time instants identified from most different sensors (including both lateral and vertical accelerometers) coincide, the real ship collision occurrence time can be confirmed. However, the unintelligibility of changes in the vertical responses during the ship collision makes the accident alarming using the vertical responses very difficult. Consequently, an effective method should be developed to detect the ship collision occurrence time, even under the condition where the data have not visibly exhibited the characteristic of ship-bridge-collision induced vibrations (i.e., sudden increases in vibration amplitudes).

4.2.2 Ship-bridge collision accident alarming method

The ship collision will induce a sudden large energy input into a bridge during a short time period. This energy tends to excite some vibration modes, which may already be motivated, but possess a very low energy under ambient loads. In considering the fact that the contribution of ship collision force is much larger than that of ambient loads in exciting these vibration modes, these modes can thus be
regarded as the ship-collision-induced vibration modes. The energy change of these modes before and after ship collision could be significant. Hence, it will be quite helpful to detect the ship collision accident if these ship-collision-induced vibration modes can be filtered out from the original data. Considering that the EMD is able to decompose a signal into several basic functions which are adaptive from the signal itself, the EMD is employed here to process the raw vertical response data in order to filter out the ship-collision-induced vibration modes. The IMFs, which are the EMD basis functions, can represent the signal components of different frequency scales. Moreover, all the IMFs are orthogonal with each other mathematically and each IMF can be viewed as a single component that has time-varying amplitude and frequency. An IMF is defined to be the function that satisfies the following two conditions (Huang et al. 1998):

(1) The number of extrema and the number of zero crossings must be either equal or differ at most by one in the whole data sets.

(2) The mean value of the envelope defined by the local maxima and the envelope defined by the local minima is zero at every point.

The procedures for extracting IMFs from a signal are introduced in the following.

(1) Identify all the local extrema, connect all the local maxima by a cubic spline as the upper envelope and then repeat the procedure for the minima to produce the lower envelope. The upper and lower envelopes should cover all the data.
(2) Designate their mean as \( m_1 \), and the difference between the signal \( s(t) \)
and \( m_1 \) as the first component \( h_1 \), that is
\[
s(t) - m_1 = h_1
\]  

Ideally, \( h_1 \) should be an IMF, since the construction of \( h_1 \) described above seems to have satisfied all the requirements of IMF. However, in reality, the existence of overshoots and undershoots can generate new extrema, and shift or exaggerate the existing ones. Therefore, a sifting process with the intention of eliminating riding waves and forcing the local symmetry about the zero-mean line is used to extract the essential scales from the data. \( h_1 \) is treated as the signal and the second step is repeated as
\[
h_1 - m_{11} = h_{11}
\]  
The sifting process will be repeated \( k \) times, until \( h_{1k} \) becomes a true IMF, that is,
\[
h_{1(k-1)} - m_{1(k-1)} = h_{1k}
\]  
Then it is designated as
\[
c_1 = h_{1k}
\]  
the first component of the data, which has the finest scale or the shortest period of the signal.

(1) Removing \( c_1 \) from the rest of the signal by
\[
s(t) - c_1 = r_1
\]  

(2) The residue \( r_1 \), which contains information on longer period components, is now treated as the new data and subjected to the same sifting process in Step (2).
The procedure is repeated for all subsequent $r_j$ s as

$$r_1 - c_2 = r_2, \ldots, r_{n-1} - c_n = r_n$$

(4.6)

The whole EMD process is stopped by any of the following predetermined criteria: either when the component $c_n$, or the residue $r_n$, becomes so small that it is less than the predetermined value of substantial consequence, or when the residue $r_n$ becomes a monotonic function from which no more IMF can be extracted. By summing up Equations (4.5) and (4.6), we finally obtain

$$z(t) = \sum_{i=1}^{n} c_i + r_n$$

(4.7)

After obtaining IMFs, the ship-collision-induced vibration modes can be identified by observing the energy change in each IMF. If a signal contains the information about a ship bridge collision accident, it will manifest itself by exhibiting a sudden increase of vibration amplitude (energy) in some IMFs. These IMFs are then identified as the ship-collision-induced vibration modes. To observe the energy change in the time domain more easily and accurately, we apply the Hilbert transform to each of the IMFs to compute the instantaneous amplitudes, which are capable of reflecting the instantaneous energy change in the time domain.

The Hilbert transform of a real-valued function $\chi(t)$, which belongs to $L^p$ space, is given by

$$H[\chi(t)] = \gamma(t) = \frac{P}{\pi} \int_{-\infty}^{\infty} \frac{\chi(\tau)}{t-\tau} d\tau$$

(4.8)

where $P$ is the Cauchy principal value. The function $\chi(t)$ and its Hilbert transform $\gamma(t)$ form an analytic signal $z(t)$ given by
where \( a(t) \) and \( \theta(t) \) represent the instantaneous amplitude and phase, respectively. They can be calculated as follows

\[
a(t) = \sqrt{x^2 + y^2}
\]

(4.10)

\[
\theta(t) = \tan^{-1} \frac{y}{x}
\]

(4.11)

After computing instantaneous amplitudes by use of Hilbert transform, the ship-collision-induced vibration modes are readily identified by observing the instantaneous amplitudes of each IMF. Then the detection of the ship-bridge collision occurrence time can be realized in the following two ways: (i) find the time instant of the sudden increase in the instantaneous energy of the ship-collision-induced vibration modes by observing the their instantaneous amplitudes; (ii) add all the ship-collision-induced vibration modes together to represent the ship-collision-induced response (SCIR) inherent in the signals and find the time instant of the sudden increase in vibration amplitude though observing the SCIR time sequence directly.

4.2.3 Application of the proposed ship-bridge collision accident alarming method

The proposed ship-bridge collision accident alarming method is applied to the measured vertical acceleration responses during the ship collision within one-hour duration, in which, the changes of vibration amplitude are not clear when observing raw data sequences in the presence of environmental uncertainty, ambient vibration and measurement noise. The data measured by the sensor AD5WV are taken as an
example to illustrate the ship collision accident detection process.

The EMD is utilized to decompose the data into 17 IMFs, and then the instantaneous amplitudes of each component are calculated by use of Hilbert transform. Figures 4.4(a) and (b) present the resulting 17 IMFs as well as their instantaneous amplitudes.
Figure 4.4(b) IMFs 9 to 17 of response signals from AD5WV and their instantaneous amplitudes
corresponding instantaneous amplitudes. It is seen from Figure 4.4(a) that five IMFs (IMFs 4 to 8) show clear sudden increase in the instantaneous amplitudes at about 20:14. This phenomenon implies that an impact loading was applied on the structure at that moment. Among different kinds of loadings that may be applied on a bridge structure, only the ship collision force can be viewed as a kind of impact loading. Therefore we can justifiably conclude that a ship collision accident occurred at about 20:14. The IMFs 4 to 8 are identified as ship-collision-induced vibration modes. The accurate ship bridge collision time is determined as 20:14:23 by observing the peak values from the instantaneous amplitudes of IMFs 4 to 6. However, the low-energy and low-frequency ship-collision-induced vibration modes (IMFs 7 and 8) exhibit several peaks in amplitudes which may be excited by uncertain environmental loadings or by the subsequent ship-bridge interactions with low energy inputs. In order to reduce the uncertainty and detect the beginning time of accident accurately, the above five IMFs are added together to represent the SCIR inherent in the signals as shown in Figure 4.5. In this figure, the ship collision is easily and accurately

![Figure 4.5 SCIR](image-url)
detected occurring at the time instant 20:14:23. A comparison between the SCIR inherent in the signals filtered out by the EMD in Figure 4.5 and the raw time-history data in Figure 4.3(b) indicates that the EMD is effective in ship-bridge collision accident alarming. The ship collision time instants identified from the lateral sensors AD9CL and AD13CL and vertical sensors AD5WV, AD7EV, AD7WV, AD9EV, AD9WV, AD13EV, and AD13WV respectively are all coincident, signaling that a ship-bridge collision accident happened at 20:14:23 on June 2, 2005.

4.2.4 Noise effect

The effect of measurement noise is investigated to examine the robustness of the proposed ship-bridge collision accident alarming method. Although the measurement noise exists in the monitoring data, it cannot be perfectly separated from the data. Therefore, here the monitoring data are regarded as the original signal and finite white noise with different intensities is added into the original monitoring data to simulate the signals contaminated with white noise of different noise levels. The noise-to-signal ratio is defined to measure the intensity of the white noise and given by the following formula.

\[
NSR = \frac{P_{\text{Noise}}}{P_{\text{Signal}}} \times 100\%
\]  
(4.12)

where \( P_{\text{Noise}} = \frac{\sum_{i=1}^{N_t} \text{Noise}^2(t_i)}{N_t} \); \( P_{\text{Signal}} = \frac{\sum_{i=1}^{N_t} \text{Signal}^2(t_i)}{N_t} \); \( t_i \) is the time point and \( N_t \) is the number of time points.

The data measured by the sensor AD5WV are taken as an example to examine the effect of noise. The original monitoring data, white noise contaminated data
Figure 4.6 Comparison of original monitoring data, white noise contaminated data with a 10% noise-to-signal ratio, and filtered SCIR from noise contaminated data by EMD: (a) original monitoring data; (b) white noise contaminated data with a 10% noise-to-signal ratio; and (c) filtered SCIR from noise contaminated data by EMD

whose noise-to-signal ratios is 10%, and filtered SCIR from the noise contaminated data by the EMD are shown in Figure 4.6. Although the ship-bridge collision accident can neither be detected accurately from the noisy data nor from the original data, the filtered SCIR successfully signals the ship-bridge collision accident. Figure 4.7 shows filtered SCIR from the white noise contaminated data whose noise-to-signal ratios is 50%, as well as the original data and noisy data. The comparison between Figures 4.7(a) and (b) reveals that the white noise obscures the original data dramatically. However, it is seen from Figure 4.7(c) that the ship-bridge collision accident can still be identified clearly. When the noise level
Figure 4.7 Comparison of original monitoring data, white noise contaminated data with a 50% noise-to-signal ratio, and filtered SCIR from noise contaminated data by EMD: (a) original monitoring data; (b) white noise contaminated data with a 50% noise-to-signal ratio; and (c) filtered SCIR from noise contaminated data by EMD.
Figure 4.8 Comparison of original monitoring data, white noise contaminated data with a 80% noise-to-signal ratio, and filtered SCIR from noise contaminated data by EMD: (a) original monitoring data; (b) white noise contaminated data with a 80% noise-to-signal ratio; and (c) filtered SCIR from noise contaminated data by EMD.

Further increases into 80% noise-to-signal ratio, the accident identification result is still very good, as shown in Figure 4.8. From the above results, we find that the proposed method is not sensitive to noise and the identification result is reliable even when the monitoring data are highly corrupted with noise.

4.3 Condition Assessment of Ship-collided Bridges

4.3.1 Condition assessment method for ship-collided bridges

As discussed in Section 4.1, although the HHT is known to be capable of capturing the instantaneous frequencies and reflecting transient energy distributions
of signals, it suffers from mode mixing problem, a major drawback of the HHT during the EMD process, which renders the decomposition of narrow band multi-component signals problematic. Some researchers have used the EEMD technique to alleviate the problem (Wu and Chung 2009; Wu and Huang 2009). However, it is still difficult to separate the vibration modes for large-scale bridges, whose vibration frequencies are very low and frequency bands are densely spaced (for example, the first 16 modal frequencies of the Jiangyin Bridge are below 0.5 Hz), by use of the EEMD based methods. In this study, a hybrid method that combines the EMD, FFT, band-pass filter and Hilbert transform is proposed to process the data in order to overcome the deficiencies of HHT.

Firstly, the ship-collision-induced vibration modes filtered by the EMD in last section are processed by the FFT, in order to identify their vibration frequency band. The frequency band between zero and the largest frequency of the ship-collision-induced vibration modes is regarded as the concerned frequency range in the condition assessment of ship-collided bridge. Then according to the identified ship collision time instant, a short segment of data covering the time period before, during, and after the ship collision is selected for condition assessment.

After determining the concerned frequency range and the short segment of data that will be used for condition assessment, the FFT and band-pass filter are combined to filter the data into narrow band signals. Because frequency bands are densely spaced for large-scale bridges, the band-pass filter should be narrow enough to separate different vibration modes. Among a number of band-pass filters, single
comb filter has a very narrow pass band. Hence, it is suitable for separating the densely spaced vibration modes. The single comb filter, and the single notch filter which is a band-stop filter with a narrow stopband, are complementary filters in the sense that their frequency responses add up to unity (Orfanidis 1996):

\[ H_{\text{notch}}(\omega) + H_{\text{comb}}(\omega) = 1 \]  \hspace{1cm} (4.13)

A second-order single notch filter can be designed as follows (Orfanidis 1996):

\[ H_{\text{notch}}(z) = b \frac{1 - 2 \cos \omega_0 z^{-1} + z^{-2}}{1 - 2b \cos \omega_0 z^{-1} + (2b - 1)z^{-2}} \]  \hspace{1cm} (4.14)

where \( \omega_0 \) is central frequency; the filter parameter \( b \) is expressible in terms of the 3-dB band width \( \Delta \omega \) (in units of radians per sample) as follows:

\[ b = \frac{1}{1 + \tan(\Delta \omega / 2)} \]  \hspace{1cm} (4.15)

The single comb filter can be determined by substituting Equation (4.14) into Equation (4.13) as

\[ H_{\text{comb}} = \frac{1 - b + (b - 1)z^{-2}}{1 - 2b \cos \omega_0 z^{-1} + (2b - 1)z^{-2}} \]  \hspace{1cm} (4.16)

The designed single comb filter can be used to filter the narrow band signal with central frequency \( \omega_0 \) and 3-dB band width \( \Delta \omega \) out of the original signal.

The central frequency \( \omega_0 \) and the band width \( \Delta \omega \) for each frequency band are determined from the results of the FFT analysis. The original data are processed by the FFT, and the peaks in the FFT spectrum are identified as the central frequencies of the corresponding frequency bands. The band width \( \Delta \omega \) is determined according to the frequency range between two neighboring peaks, i.e., it should be small enough to separate the two neighboring vibration modes. Then the
single comb filters designed for each frequency band are applied to the data respectively, the resulting filtered signals are regarded as the separated vibration modes.

Then Hilbert transform is applied to the separated narrow band vibration modes for extracting their instantaneous frequencies. The instantaneous frequency is defined as the time derivative of the instantaneous phase in Equation (4.11), as follows:

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

(4.17)

The \( k \)th filtered signal is denoted as \( s_k \), and its instantaneous amplitude and frequency are denoted as \( a_k \) and \( \omega_k \), respectively. The transient energy distributions of the measured dynamic responses covering the time period before, during and after the ship collision in the time-frequency domain can be reflected by the Hilbert spectrum in terms of \( a_k \) and \( \omega_k \), i.e.,

\[
H(\omega, t) = \sum_{k=1}^{m} a_k(t) \exp(i \int \omega_k(t) dt)
\]  

(4.18)

4.3.2 Application of the proposed condition assessment method for ship-collided bridges

Again, the monitoring data from the Jiangyin Bridge during the ship collision accident are analyzed. After identifying the ship collision occurrence time, it is found by carefully examining the data that while the signals from some sensors exhibit several abnormal peaks and spikes in one-hour data, they are normal during the short period around the ship collision time. Therefore the short-period data
measured from all the accelerometers prior to, during and posterior to the ship collision time can be included in the condition assessment. Taking the measured response data from the sensor AD5WV as an example, the signal processing procedure is briefed as follows. The FFT spectra of ship-collision-induced vibration modes (IMFs 4 to 8) shown in Figure 4.4(a) are given in Figure 4.9. It is observed from Figure 4.9 that the largest frequency of the ship-collision-induced vibration modes is 1.8 Hz. This implies that most of the vibration modes excited by the ship collision have frequencies within the range between 0 and 1.8 Hz. Hence, this frequency range is regarded as the concerned frequency range in the condition assessment of the ship-collided bridge. The ship collision occurrence time has

![Figure 4.9 FFT spectra of ship-collision-induced vibration modes](image-url)
already been identified at 20:14:23, and therefore the data used for the condition assessment is selected in the three-minute period from 20:14:00 to 20:17:00, which is thought to be able to cover the time period before, during, and after the ship collision. The three-minute data are firstly processed by the FFT, and the peaks within the concerned frequency range in the FFT spectrum are identified as the central frequencies of the corresponding frequency bands. Figure 4.10 shows the FFT spectrum of the three-minute data from the sensor AD5WV and the peaks within the concerned frequency range. Subsequently the single comb filter is used to filter the signals into several narrow band components with their central frequencies being the identified peak values in Figure 4.10(b). Finally, the narrow band components are processed by Hilbert transform, and the instantaneous frequencies

![Figure 4.10 FFT spectrum of three-minute data from AD5WV: (a) FFT spectrum of frequency range from 0.01 to 25 Hz; and (b) Peaks within range between 0 and 1.8 Hz](image-url)
are extracted and the energy distribution in both time and frequency scales is expressed by the Hilbert spectrum. The Hilbert spectrum of the data from the sensor AD5WV in the color map format is shown in Figure 4.11. The different colors in the figure represent the different amplitudes (energies). It is observed from Figure 4.11 that the Hilbert spectrum is able to clearly reveal the detailed changes of instantaneous frequencies and provide the information about energy variation of the measured dynamic response in the observation period and concerned frequency range.

![Hilbert spectrum of response data from AD5WV](image)

Figure 4.11 Hilbert spectrum of response data from AD5WV

In Figure 4.11, before the time instant 20:14:23, the energy in the high frequency range between 0.4 and 1.8 Hz is negligible compared with the energy in
the low frequency range below 0.4 Hz, suggesting that the low frequency modes are dominant in the response data. Since 20:14:23, the energy has suddenly increased significantly in the whole concerned frequency range, with the greatest energy values distributing in the region around 0.5 Hz. The time instant of sudden energy change coincides quite well with the previously identified ship collision time instant. This indicates that the Hilbert spectrum has a fine time resolution and is capable of capturing the ship collision time information. Although the FFT spectrum illustrated in Figure 4.10 also shows the greatest energy component at about 0.5 Hz, it was unable to give any local time information except exhibiting an overall energy distribution in the frequency domain. From the ambient vibration identification, the frequency of 0.5 Hz is actually the 16th vibration mode of the Jiangyin Bridge with a symmetric vertical modal configuration. This indicates that the participation of the 16th vibration mode has become dominant in the measured responses since the ship collision. Also, other higher-order modes in the frequency range from 0.4 to 1.8 Hz, which cannot be observed before the ship collision, are seen clearly since the ship collision, suggesting that the ship collision impact force tends to excite the higher order vibration modes.

In structural damage detection, two phenomena may contribute the sign of commence to structural deterioration. First, the reduction in modal frequencies may indicate the loss of structural stiffness. Second, the spreading of vibration energy to the neighboring modes as an energy intensity decrease suggests the presence of structural nonlinearity (Pines and Salvino 2002). After carefully inspecting Figure
4.11, it is found that the instantaneous frequencies of the vibration modes with relatively higher energy (such as the modes with frequencies of 0.20, 0.26, 0.40, 0.50, and 0.56 Hz) remain almost constant within the observation duration from 20:14:23 to 20:17:00, except for some frequency modulations occurring at the beginning of the ship collision. This may result from the low energy input when the ship just contacted the bridge. Meanwhile, the instantaneous frequencies for the other modes are constant in general with small modulations, which may result from the relatively small energy distribution in these components and the noise effect. The generally constant instantaneous frequencies indicate that the bridge structural stiffness remains unchanged during the ship collision accident. In the time domain, the energy intensity is changing: in the high frequency range between 0.4 and 1.8 Hz, it increases during approximately the first 60 seconds of the ship collision and then decreases slowly in the remaining observation duration; while in the low frequency range below 0.4 Hz, it successively increases for about 100 seconds since the beginning of the ship collision and then begins to diminish gently. However, in the frequency domain, the energy intensity distribution of each mode is within the narrowband, without any energy shift to other bands. This indicates that no structural nonlinearity appears. From both the instantaneous frequency information and energy distribution information, it is concluded that the bridge did not suffer from damage during the ship collision accident. The results obtained from the other accelerometers are similar to the above and therefore not presented here. The damage detection results given here coincide with those obtained by a novelty
detection technique which is based on auto-associative neural networks (Zhou et al. 2006).

4.3.3 Comparison with wavelet scalogram

Compared with the results of the Hilbert spectrum, the wavelet scalogram generated by the Morlet wavelets with a central frequency of 3 Hz (several values are tested as central frequency, and the wavelet with the central frequency of 3 Hz performs best, with the best time-frequency resolution) exhibits a different energy distribution style in the time-frequency domain, as shown in Figure 4.12. The wavelet scalogram can be used to roughly identify the time instant of sudden energy increase due to the ship collision in the time domain; however, the smearing of the precise time instant of sudden energy increase in the low frequency range is shown. Although the concentration of energy appears at about 0.5 Hz similarly as the Hilbert spectrum, the energy leakage to the neighboring bands is obvious. In

Figure 4.12 Morlet wavelet scalogram of response data from AD5WV
addition, the energy distribution in other bands is much lower. This is because the wavelets have a tendency to concentrate their largest coefficients at the dominant frequency components of signals (Kijewski-Correa and Kareem 2006). While the Hilbert spectrum appears in a skeleton form with emphasis on the frequency variation, the wavelet scalogram results in a smoothed energy contour map without showing the instantaneous frequencies clearly. It seems to be difficult to ascertain the presence of damage in the wavelet scalogram.

4.4 Summary

A set of HHT based approaches is developed as the signal analysis tool to process measured bridge responses during ship collision. The proposed HHT based ship-bridge collision accident alarming method is adopted to detect the occurrence time of ship collision accident for accident alarming. The processing results of the in-situ measurement data acquired from the instrumented Jiangyin Bridge indicate that the method effectively increases the accuracy of accident alarming. When it comes to accessing the condition of ship-collided bridge, the original HHT exhibits some problems. In order to overcome the deficiencies of the original HHT, a hybrid method that combines the EMD, FFT, band-pass filter and Hilbert transform is developed to interpret the condition of ship-collided bridges. The case study of the Jiangyin Bridge shows that the resultant Hilbert spectrum is effective in ship-collided bridge condition assessment, in comparison with the wavelet scalogram and the FFT spectrum.
CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Rather than causing the collapse of whole bridges, most of the ship-bridge collision accidents usually lead to invisible structural damage, which is well hidden behind an apparent structural integrity condition and can bring about hidden danger threatening the bridge safety. As a result, assessment of structural damage and change in structural condition immediately after a ship collision event is quite important when deciding whether to close a bridge to traffic and when planning consequent bridge retrofitting. The research in this study is devoted to developing specific methods for detecting the damage and assessing the condition/safety of bridges after experiencing ship collision based on the monitoring data measured by on-line SHM systems installed on bridges. The aim of the research is to (i) propose a sensor placement optimization method targeting post-collision damage detection for bridges subject to ship collision; and (ii) develop a set of approaches for ship-bridge collision accident alarming and condition/damage assessment for post-collision bridges using HHT based techniques.

In this study, an information-entropy-based OSP method is proposed to deal with the specific OSP problem for damage detection of bridges subject to ship collision. This approach takes into account the unknown or uncertain ship collision position in the sensor placement design stage and breaks through the limitations of the sensitivity-based and information-entropy-based OSP methods for the purpose of
damage detection in previous investigations, when applied to large-scale structures. Additionally, a set of HHT based approaches is developed as the signal analysis tool to process measured bridge responses, in order to detect/alarm ship-bridge collision accidents and detect/assess the structural damage incurred during ship-bridge collision.

i. Development of a sensor placement optimization method targeting post-collision damage detection for bridges subject to ship collision

No investigation has been reported on dealing with the OSP problem for post-collision damage detection of bridges. The contribution of this study lies in proposing an OSP method targeting post-collision damage detection for bridges subject to ship collision, so that the measured data are most informative for damage detection. The feasibility of the method has been demonstrated by numerical studies using the cable-stayed TKB. In addition, the effectiveness of the method has been examined and assessed under different damage scenarios through numerical studies. The specific findings and conclusions are as follows:

1. An important aspect of this specific OSP problem is concerned with the unknown or uncertain ship collision position in the sensor placement design stage, which has not yet been addressed in previous investigations. The proposed OSP method handles the uncertainty of ship collision position by formulating the OSP problem as a multi-objective optimization with each objective representing one possible ship-bridge collision scenario. The final optimal solution is a compromise sensor configuration trading-off the qualities of information for all
scenarios and is thus able to provide almost equal information for all the ship collision scenarios without giving any preference to a specific scenario. The numerical example demonstrates that the resulting optimal sensor configuration is effective under different ship collision scenarios in the sense that the optimally selected vertical DOFs are able to provide much more damage information than the longitudinal and transverse DOFs and capture the damage information from most of the local sensitive regions defined according to the damage information in the vertical direction.

2. The sensitivity-based and information-entropy-based OSP methods for the purpose of damage detection in previous investigations define the damage measurability of a sensor location based on the sensitivity matrix with respect to the model parameters of each element. However, for large-scale structures like cable-stayed and suspension bridges, it is very difficult to do so due to the huge number of structural elements. To overcome the problem, the DMM, which can be used to describe the input-output behavior of a bridge, is proposed to significantly reduce the number of parameters through a two-step parameterization.

3. The sensitive regions with respect to the change in DMM parameters can be identified by clustering analysis. With the identified sensitive regions, the sensor configuration can be finally determined by randomly reserving a few selected DOFs in each sensitive region and keeping all the other selected DOFs in non-sensitive regions. It guarantees a redundancy of sensors deployed at the
most informative regions, and thus sufficient information can still be obtained even in case some of the sensors are malfunctioned.

4. The proposed method uses the acceleration responses obtained from the FE analysis, and is therefore applicable in practice to determine the sensor placement prior to field testing.

5. It leaves some freedom to determine the critical elements of a bridge which are in need to be monitored. This makes the method adaptable in different applications.

6. It is worthwhile mentioning that in the regions with distinct seasonal changes, the seasonally environmental change, such as humidity, wind, and most important, temperature, can cause the seasonal changes in the monitoring data and in measured modal frequencies by up to 18% (Peeters and De Roeck 2001), and even sensor damage in hostile conditions. In these regions, in order to ensure the security of the on-line SHM system, the sensor placement should take the seasonally environmental effect into consideration by selecting the sensors which are insensitive to environmental changes and able to work even under the hostile conditions (Dauderstädt et al. 1995; Kapser et al. 2003; Tian et al. 2004). However, the effect of seasonal change on monitoring data is small in Hong Kong, with the variation in the nature frequencies being about 6.7% or less, due to mild climate in this area (Ni et al. 2009). Also, the ship-bridge collision induced responses are transient signals, the seasonally environmental changes do not affect the measurement of this kind of responses. Therefore, in this study, the
seasonal effect on sensor placement is not a major concern.

ii. Development of a set of HHT based approaches for ship-bridge collision accident alarming and condition/damage assessment of collided bridges

The contribution of this study includes the development of a HHT based ship-bridge collision accident alarming method and a hybrid condition/damage assessment method that combines the EMD, FFT, band-pass filter and Hilbert transform; the former lays the base for the latter, since a ship-bridge collision accident should be detected and alarmed before the condition assessment of the collided bridge is conducted. The measured acceleration responses of the suspension Jiangyin Bridge during a ship collision event are used to demonstrate the proposed procedure and testify its performance in alarming the ship-bridge collision accident and assessing the ship-collided bridge condition. The instantaneous frequencies and transient energy distributions of the collision-induced responses reflected by the resultant Hilbert transform are analyzed in the time-frequency domain, and are compared with the traditional power spectral densities obtained by the FFT and those by the wavelet transform. The specific findings and conclusions are as follows:

1. Although the ship collision accident in the Jiangyin Bridge can be detected by observing a sudden increase in the amplitudes of raw data of the bridge responses in the lateral direction, the changes of vibration amplitudes in the vertical direction are not clear when observing raw data sequences in the presence of environmental uncertainty, ambient vibration and measurement noise.
2. A further examination of the lateral response data measured during the ship collision accident of the Jiangyin Bridge shows that three out of the total five lateral responses exhibit several abnormal peaks and spikes especially after the ship collision accident occurred, which may be due to heavy trucks passing through or the emergency action taken in the accident. If the exact time of the accident is completely unknown, these alias peaks and spikes may induce a false alarm. Hence, just using the lateral responses to alarm the accident is not reliable enough given the susceptibility of the accelerometers to ambient effects.

3. It is found that the vertical accelerometers are relatively more reliable than the lateral ones, therefore the vertical responses should also be utilized in accident alarming. Conceptually, if accident happening time instants identified from most different sensors (including both lateral and vertical accelerometers) coincide, the real ship collision occurrence time can be confirmed.

4. By use of the proposed ship-bridge collision accident alarming method, the ship-bridge collision occurrence time can be detected by observing the time instant of the sudden increase in the instantaneous amplitudes of the ship-collision-induced vibration modes, or finding the time instant of the sudden increase in vibration amplitude though observing the SCIR time sequence directly. The ship collision occurrence time in the Jiangyin Bridge has been accurately detected using the vertical responses, the changes of vibration amplitudes of which are not clear when observing raw data sequences.

5. The proposed ship-bridge collision accident alarming method is not sensitive to
noise and the identification result is reliable even when the monitoring data are highly corrupted with noise.

6. Although the HHT is known to be capable of capturing the instantaneous frequencies and reflecting transient energy distributions of signals, it suffers from mode mixing problem, a major drawback of the HHT during the EMD process, which renders the decomposition of narrow band multi-component signals problematic. The proposed hybrid condition/damage assessment method successfully overcomes the deficiencies of the original HHT through combining the EMD, FFT, band-pass filter and Hilbert transform. The resultant Hilbert spectrum is able to clearly reveal the detailed changes of instantaneous frequencies and provide the information about energy variation of the measured dynamic response in the observation period and concerned frequency range.

7. The resultant Hilbert spectrum has a fine time resolution and is capable of capturing the ship collision time information. Although both the FFT spectrum and Hilbert spectrum show the same greatest energy component, the former is unable to give any local time information except exhibiting an overall energy distribution in the frequency domain, while the latter could indicate ship collision time instant accurately. The wavelet scalogram can be used to roughly identify the time instant of sudden energy increase due to the ship collision in the time domain. However, the smearing of the precise time instant of sudden energy increase in the low frequency range is shown.

8. The resultant Hilbert spectrum can be used to detect the change of structural
stiffness and nonlinearity by providing the information of instantaneous frequencies in the time domain and energy intensity distributions in the frequency domain. While the Hilbert spectrum appears in a skeleton form with emphasis on the frequency variation, the wavelet scalogram results in a smoothed energy contour map without showing the instantaneous frequencies clearly. The latter also suffers from the energy leakage problem. It seems to be difficult to ascertain the presence of damage in the wavelet scalogram.

### 5.2 Recommendations

In this MPhil study, a sensor placement optimization method targeting post-collision damage detection for bridges subject to ship collision and a set of approaches for ship-bridge collision accident alarming and condition/damage assessment of collided bridges using HHT based techniques have been proposed. However, there are still some limitations on the developed methods and unsettled issues regarding the ship-bridge collision problem, which are worth addressing in the future: (i) the proposed OSP method targeting post-collision damage detection for bridges subject to ship collision have been testified by numerical studies only and still need to be verified using the real-world data; (ii) The proposed collided bridge condition/damage assessment approaches have been used to analyze the monitoring data of the Jiangyin Bridge in one scenario only and it is desirable to apply the approaches to more data measured in different ship-bridge collision scenarios; (iii) ship collision loads to be used in design have not been well understood yet, more investigations are needed in accurate identification of ship
impact forces using measured response data; and (iv) the OSP method may need to be extended to make the measurement data also informative for estimating the impact force if the purpose of impact force identification is considered.

i. Validation of the effectiveness of the proposed OSP method targeting post-collision damage detection of bridges using real-world data

The proposed method can provide the owner of the TKB with more reasonable sensor placement designs in the future SHM system upgrading project. The effectiveness of the method in field application could then be further validated when monitoring data measured by the optimized sensor system during ship-bridge collision accidents are available. It is also desirable to apply the method for different bridges which are built cross waterways.

ii. Application of the proposed collided bridge condition/damage assessment approaches to different ship-bridge collision scenarios

The proposed approaches have been used to analyze the data measured in a ship-bridge collision accident of the Jiangyin Bridge. However, the accident is a small one which does not cause damage to the structure. When the monitoring data are available, it is desirable to apply the methods in the scenarios where the structural nonlinearity or damage is caused by the accident. Then the effectiveness of the ability of condition/damage assessment will be further testified.

iii. Identification of ship impact forces

The existing researches on evaluation of ship impact forces are based on energy analysis and FE simulations, however, the recently developed SHM systems provide
another alternative for estimating the ship impact forces. The structural responses during ship collision can be measured by SHM systems automatically for the instrumented bridges and the data can be used to identify ship impact forces. The most common approach to identify the impact force is to calculate the force inversely by the FRF matrix and the measured structural responses using least square method. The method is effective in reducing the random errors, however, its accuracy is often hindered by the relatively large modeling error and the inversion of an ill-conditioned FRF matrix at the resonance frequencies. Liu and Shepard (2005) investigate the method to deal with the problem in the inversion of an ill-conditioned FRF matrix by using the truncated singular value decomposition filter and the Tikhonov filter. For the problem caused by the relatively large modeling error, unfortunately, no satisfactory algorithm has been proposed yet. The model updating techniques are suggested to be used in conjunction with the inverse force identification method to reduce the modeling error, improve the accuracy of the FRF matrix and thus increase the accuracy of force identification.

iv. Extension of the OSP method for the purpose of ship impact force identification

If the purpose of impact force identification is considered, it is desirable that the measurement data are also informative for estimating the impact force. The transfer function between the impact force and the response varies with the location. The modules of the transfer function can become very small at some locations at a certain frequency. The responses at those locations contain little information about
impact force at that frequency. However, at some other locations, the responses may contain much information about impact force at that frequency. Therefore, for the purpose of impact force identification, the sensors should be placed such that the obtained information about impact force is as more as possible at the whole frequency range. This goal may be achieved by defining an index based on FRF to measure the content of information about impact force at the whole frequency range and maximizing the index in the optimization process.
REFERENCES


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