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Structural Health Monitoring of Critical Load-Carrying Members

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The structural integrity of transportation infrastructure relies greatly on the condition of critical load-carrying members. Many in-service highway bridges use primary beams with welded cover plates that provide greater flexural capacity in regions of high-applied moment. The cyclic loading pattern of a bridge induces fatigue damage starting from the toe of the welds due to high stress concentrations. The crack propagates well into the beam’s web, reducing flexural capacity. This behavior is present in other typical connection details where stress concentrates greatly, such as floor-beams to main truss connections, diaphragms and cross-bracing connections, copied and cut-short beam ends, and stringer-to-floor beam connections. Main tension elements in cable-supported bridges also suffer fatigue and corrosion damage, which can be greatly challenging to detect with conventional visual inspection methods. This damage can reduce tension capacity sufficiently to require early replacement of stay cables.

Structural health monitoring (SHM) methods provide an objective and quantitative option for structural assessment. Cracks in steel members can be monitored at a low cost with commercial radio frequency identification (RFID) tags. An RFID-based crack sensor has been developed and experimentally assessed. This crack sensor uses wireless technology and a simple damage extraction feature to characterize crack width and identify initial crack formation. A guideline for sensor deployment has been established considering its damage sensitivity on metallic surfaces. For tension monitoring in cable structures, a new smart wireless system capable of using ambient vibration measurements for cable tension estimation has been developed. The same vibration measurements collected for cable tension estimation can be used to identify damage using a flexibility-based damage identification method. This damage identification algorithm has been modified to automate calculation and improve accuracy in damage quantification and location. These developments present a relevant advancement in simple, low-cost, and reliable SHM that provides practical information to bridge owners for intelligent decision-making.
Structural Health Monitoring of Critical Load-Carrying Members

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Structural Health Monitoring of Critical Load-Carrying Members

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Chapter 1 Introduction

The improved performance and safety of civil structures is the primary goal of civil engineers. Transportation infrastructure is one of the most critical engineered products that serve our society and allow it to operate and grow. Yet, every day in the United States 188 million trips are made over structurally deficient bridges and 9.1% of the nation’s bridges are classified as structurally deficient [1]. A structurally deficient bridge has a deck, superstructure or substructure in poor condition or its load-carrying capacity is significantly below minimum standards [2]. In face of a natural or man-made disaster, this poses a serious threat to the execution of emergency respondent logistics, as the failure of such structures could disconnect communities from the necessary provisions and services that must remain accessible after a disaster. It has been observed that damage begins at the material level, grows and later spreads to the component level [3]. The structural integrity of bridges relies firstly on the condition of critical load-carrying elements; thus, these members require the most immediate attention. Cables in cable-supported bridges and steel girders with welded connections in highway bridges are examples of critical structural elements that accumulate damage with great ease.

Greater concern arises when it is considered that the cyclic nature of bridge loading accelerates the damage of these critical load-carrying elements dramatically. Cyclic loading can produce fatigue, which occurs progressively and locally, leading to sudden failure below the yield stress limit. Traditionally, highway bridges use various types of steel-welded beams that are susceptible to these effects. The National Cooperative Highway Research Program [4] noted that fatigue crack propagation occurred as cracks initiated from the toe of fillet or groove welds because of high stress concentration due to discontinuity and residual tension stress. Thus, steel-welded beams are more susceptible to fatigue crack formation than rolled beams, and this requires closely monitored information of stress level by frequent visual inspection and field measurement. In addition to fatigue damage, corrosion is also significant in cable structures due to long-term service. It has been found that the exposure of cables to
high levels of moisture can result in severe corrosion, reducing cable diameters up to 30% [5]. This translates into a loss in bending stiffness of up to 76%, and hence a significant reduction in loading capacity and vibration resistance.

In response to these problems, structural health monitoring (SHM) methods have been developed for objective and automated detection of damage and structural assessment to supplement visual inspection. SHM can be carried out as a long-term strategy where information on the structure’s ability to continue operating is evaluated in light of inevitable aging and damage accumulation. The process involves the observation of the structure over time using periodically spaced measurements, extraction of damage sensitive features from these measurements and a statistical analysis of these features to determine the current state of the system [6]. Rapid condition screening is oftentimes used to provide real-time and reliable information about system performance during and after an extreme event [3]. The statistical analysis of extracted damage sensitive features helps discriminate between features from the damaged and undamaged states of the structure in question [7]. Statistical models help answer questions regarding the existence, location, type, and extent of damage. Thus, SHM systems can provide dependable information on structural status for decision-making.

To address crack detection in crack-prone areas in steel girders, passive radio frequency identification (RFID) antennas have been studied to provide a reliable and simple method. Electrical engineers have developed in-house RFID antenna-based sensors for crack detection [8–11]. However, the assembly of these RFID-based sensors is costly given the typical large deployments needed in civil engineering applications. Moreover, the operation of these developments can be complex, requiring considerable post-processing for damage feature extraction and, in some instances, external power. Finally, not all RFID-based crack sensing systems developed in recent years have been tested for detection of propagating cracks on metallic surfaces. Commercially available RFID technology is more viable due to the reduced unit cost in comparison to in-house developments. In consequence, commercial passive ultra-high frequency (UHF) RFID tags have been used to develop the first low-cost, wireless crack sensor that is capable of detecting propagating cracks on metallic surfaces [12–14]. Analyses of the
sensitivity to damage of said sensors and the establishment of a guideline for deployment regarding assembly, preferred orientation, mounting procedure, and configuration for reasonable results are presented in this dissertation.

The loading condition of stay cables is typically the most important piece of information for condition assessment in cable-supported bridges. Changes in loading condition of cables can be due to changes in external loading, cable relaxation, or damage existence, whether on the member in question or in other members. Cables can also suffer changes in loading condition due to the cyclic loading caused by passing vehicles and natural hazards. Hence, a system capable of revealing damage location and severity to update load-carrying capacity would be advantageous. Tension is usually determined with lift-off tests, strain gauges or vibration measurements. Lift-off tests are complex and require extensive on-site intervention by experts using sophisticated equipment. Strain gauges that are exposed to harsh environmental conditions, such as humidity and freeze-thaw cycles, can corrode. Moreover, strain gauges are challenging to install and require direct contact with cable strands, which are not always exposed due to sheathing protection that is commonly used in cables to protect against corrosion.

On the other hand, vibration-based tension estimation can provide timely information on loading conditions effectively without traffic interruptions. The same vibration signals used to determine tension can also be used to determine damage condition via flexibility-based damage identification methods. This can thus save data storage space and power consumption of the smart sensing system, and it can also expedite processing. Vibration-based damage identification methods use acceleration measurements to extract damage-sensitive features, such as modal information via spectral analyses. Flexibility-based methods are reported to be sensitive to damage [15] and are also efficient in estimating damage conditions using only a few lower modes [16]. The automation of such a combined system expedites the SHM of structures that require immediate attention after an extreme event or whose traffic interruption results in serious delays and economical losses. Comprehensive systems consisting of intercommunicating independent parts would make monitoring of loading and other structural conditions faster and more convenient in financial and safety aspects. Single-board microcomputers can provide the computational
capacity and hardware options to include sensors and communication routes to trigger sensing, analyze data, broadcast results, and receive user commands to perform tension monitoring and damage detection functions on a stay cable. This allows the inclusion of multiple SHM modules to provide more complete health assessments for improved decision-making. A tension-estimating and damage identification system is presented in this dissertation, which can be expandable to include other SHM features, such as temperature monitoring and traffic counting. Furthermore, the SHM assessment can be performed automatically and remotely. This requires minimal human intervention and reduces the necessity of service interruption.

This dissertation presents a sensing system to characterize damage and loading conditions using vibration-based methods and a novel RFID-based crack sensor. A study on key components to develop an effective structural cyber-physical system (SCPS) for efficient SHM and control of structures is presented in Chapter 3 to provide a long-term goal of SHM. A new paradigm for the development of SCPSs in transportation and building structures is also presented in this chapter. As part of the development of a damage-characterizing sensing system that can be applied for cable structures, the experimental validation of a flexibility-based damage detection method capable of detecting, locating and quantifying damage is presented in Chapter 4. Chapter 5 shows another feature of this sensing system, which consists of a cable tension-estimating module programmed in a microcomputer using vibration measurements collected by peripheral sensors. Chapter 6 contains the design, development, and initial damage sensitivity assessment of the low-cost, wireless RFID-based crack sensor. Chapter 7 contains a refinement of design of the RFID-based crack sensor and additional experimental validation for monitoring crack propagation on metallic surfaces. Chapter 8 presents final concluding remarks on the work presented in this dissertation and a vision for future work.
2.1 Cyber-physical systems

A CPS is a confluence of embedded systems, real-time systems, distributed sensor systems and controls whose operations are monitored, coordinated, controlled and integrated by a computing and communication core [17]. A CPS bridges the virtual world of computing and communications with the continuous physical world using interconnected processing elements in wired or wireless networks connecting smart sensors to actuators. Some developments that have contributed to the implementation of CPSs are the availability of low-cost, small smart sensors; the computing capacity of low-cost, reduced-size microcontrollers; wireless communication; abundant internet bandwidth; and improvements in energy harvesting methods. Challenges that have been identified in any general CPS application include: the ability of computing components to overcome uncertainties inherently introduced by the physical system and its environment; synchronization across time and space between collection, computation, communication, and actuation components; robustness and tolerance of the system to component failure, either in the physical or virtual domain; development of smaller and more powerful actuators; and merging of time-based systems with event-based systems for feedback control. CPSs have been applied in medical devices, aerospace systems, transportation vehicles, defense systems, robotic systems, process control, factory automation, emergency management, and environmental control [17,18,27–31,19–26].

Several studies present CPS design approaches to civil engineering applications in SHM, structural control, and combined situations. Real-time hybrid testing presents a challenging CPS where physical and computational components must be perfectly synchronized at run-time in order to achieve reliable results. Huang et al. [32] evaluate the efficiency of a middleware architecture to maintain predictable timing between all physical and virtual components. Another study presents the use of CPSs to monitoring temporary structures for the improvement of safety in the construction industry [33]. One CPS design approach has been developed to satisfy the health monitoring (i.e. physical) requirements and the constraints imposed by a WSN (i.e. virtual component) [34]. The limitations to a centralized network
architecture are apparent: data can only be collected from a reduced number of nodes in a reasonable time frame, which results in the detection of only the most severe damage. This means that a timely detection of structural failure resulting from extreme events, such as an earthquake or an explosion, is not possible [35]. Since WSNs incur in high-energy consumption and long delays when sensors are used as simple data collection devices, a multi-level computing architecture is proposed to selectively activate additional sensors only in the damaged regions, allowing much of the network to remain asleep. This is accomplished by using a hierarchical decentralized system consisting of grouping nodes into clusters. Cluster members collect raw data from their accelerometers and transform their data into the frequency domain through Fast Fourier Transforms and power spectrum analysis. This information is communicated to the cluster head motes, where cross-spectral density and singular value decomposition is carried out to extract the structure’s mode shape vector and communicate it to the base station. The current flexibility matrix is calculated at this level and used to determine the existence and location of damage. The approach is tested using the Intel Imote2 platform with TinyOS software on a cantilever beam with single damage and a simulated truss with multiple damage locations and intensities. Although this study addresses the issues presented by the limitations of WSSs as physical constraints, it does not include any command computed in a virtual space to affect a physical component.

A type of CPS of great interest is the wireless structural control (WSC) system. This type of CPS uses a feedback control loop to influence the dynamic response of structures using sensor data collected through WSNs. As such, WSC systems play a crucial role in protecting civil infrastructure in the event of earthquakes and other disasters. Unfortunately, since WSC systems are so expensive and time-consuming to deploy, most research performed on them has been on laboratory-scale structures. Such is the case of the WSC system tested by Swartz and Lynch [36], where embedded steady-state Kalman estimators are used to minimize wireless communication in a seismically excited laboratory-scale six-story building. Because of this testing limitation, the delays and data losses that would be expected to occur in wireless networks deployed on large civil structures is not captured and so has not been exhaustively addressed. This problem has been partially resolved by developing a Wireless Cyber-Physical Simulator (WCPS)
that combines realistic simulations of WSNs and structures [37]. WCPS integrates Simulink to represent structural system dynamics and the controller with TOSSIM to simulate the WSN based on realistic wireless link models. The interfaces between the Simulink model and TOSSIM are the Interfacing Block and the Data Block, two MATLAB embedded functions in Simulink (see Figure 2.1). The WCPS has been used to develop a WSC benchmark problem for an active mass driver [38]. This benchmark problem provides a method to evaluate wireless control design issues such as network-induced delay, data loss, available sensor measurements, and measurement noise.

![Figure 2.1. Component architecture of the Wireless Cyber-Physical Simulator [37].](image)

In a strict sense, structural control strategies are CPSs since physical information is collected and used to determine physical actions in a cyber realm. However, the complexity and cross-domain communication between several networks for intelligent decisions that characterizes a CPS is not present in these systems. The delays incorporated into a more complex wireless, networked control systems are being studied and tended to with improvements to communication protocols [39]. In order to improve stability and performance (i.e. minimize packet loss and time-varying delays), a passivity-based architecture for a robotic system has been designed and tested [40]. The use of wireless networks for control represents a significant step towards the incorporation of global and component status into the controlling algorithm.
Some studies have commenced integration between SHM and controlling systems in civil structures. The Guangzhou New TV Tower (GNTVT), also known as the Canton Tower, is the most heavily instrumented super tall structure in the world. Its complicated SHM system was designed and implemented by the Hong Kong Polytechnic University for in-construction and in-service monitoring. The integration of in-construction and in-service monitoring strategies allows the establishment of a dynamically calibrated baseline model, a model that updates modal information at various stages of construction until completion [41]. This type of baseline model eases computational effort when substructure techniques are used. The GNTVT has inspired several investigations and developments that include new monitoring frameworks to improve wireless communication distance [42], improvements on sensor placement [43], evaluation of vibration-based SHM and damage detection methods [44,45], methods to eliminate noise from vibration responses [46], deformation monitoring [47], and modal parameter identification and updating for high-rise structures [48].

The SHM system consists of six modules: a sensory system, a data acquisition and transmission system, a data processing and control system, a data management system, a structural health evaluation system, and an inspection and maintenance system (see Figure 2.2). Sensors collect data on loading sources, structural response and environmental conditions. The on-line condition evaluation system compares measurement data with design values, FEM analysis results, and predetermined patterns and thresholds for quick assessment. The off-line condition evaluation system consists of damage diagnostic and prognostic algorithms for a more detailed health and safety assessment.

Information from the SHM system is used to verify the effectiveness of a wind vibration control system. The hybrid control system consists of two tuned mass dampers coupled with two active mass dampers and two tuned mass dampers suspended at different heights. The control system is activated by signals from anemometers and a seismograph. Ad hoc transducers provide feedback to the vibration control algorithm. These ad hoc signals are also transmitted to the monitoring center for comparison with structural response signals to detect possible faults in the ad hoc transducers. This technique of
redundancy and cross-domain networking is a step closer to the integration of monitoring and actuation networks, which characterizes a CPS.

Another study that integrates SHM and structural control systems proposes an energy harvesting, cable tension-estimating and vibration-controlling strategy (see Figure 2.3) [49]. In order to supply energy to wireless sensors and an MR damper, an electromagnetic induction device is used. It was found that the electromagnetic induction device generated sufficient energy to operate an Imote2 wireless sensor node twice per day for a month. This translates into enough power to operate 45 Imote2 sensors for a one-time sensing. Free vibration tests were also performed to evaluate cable tension estimation and vibration controlling capabilities. It was found that electromagnetic field signals provided similar power spectral information as acceleration signals to estimate cable tension with 2.5% error. The MR damper also provides damping 20% larger than a passive optimally tuned device.

Figure 2.2. Structural health monitoring scheme at the GNTVT [41].
Although this multi-functional system and the GNTVT systems are irrefutably combining SHM and structural control functionalities on the same structures, they do not address several issues related to the development of an SCPS. The interconnectivity required between SHM and structural control sensing networks and virtual components that define an SCPS has not been addressed in any study to this date. Information sharing between monitoring and control systems is necessary to continually validate effectiveness of both systems and to provide additional health assessment and control criteria for smarter decisions. Full-scale implementations of WSC systems still need to be performed. Benchmark problems that incorporate SHM evaluation criteria into controlling algorithms need to be developed. The following section will define what an SCPS should consist of, what aspects have already been addressed and which problems still need to be researched.

2.2 Structural health monitoring

SHM can be carried out as a long-term strategy where information on the structure’s ability to continue operating is evaluated in light of inevitable aging and damage accumulation. The process involves the observation of the structure over time using periodically spaced measurements, extraction of damage sensitive features from these measurements and a statistical analysis of these features to determine the current state of the system [6]. Rapid condition screening is oftentimes used to provide real-time and reliable information about system performance during and after an extreme event [3].

Figure 2.3. Multi-functional cable damping, tension-estimating and energy harvesting system [49].
The statistical analysis of extracted damage sensitive features helps discriminate between features from the damaged and undamaged states of the structure in question [7]. Statistical models help answer questions regarding the existence, location, type, and extent of damage. They can also offer information on the structure’s prognosis [50–55]. When data are available from both the undamaged and damaged states, the statistical pattern recognition algorithm falls into a general classification known as supervised learning. Unsupervised learning occurs when the algorithms are applied to data that does not contain damaged samples [56]. Unsupervised learning environments can only provide information on existence and location of damage. The main advantage of supervised learning is that damage type and extent and prognosis can be determined due the availability of correlated measured features [57–59].

Vibration-based monitoring methods allow a versatility that other nondestructive evaluation methods may not. The basic premise of most vibration-based damage detection methods is that damage will alter the stiffness, mass, or energy dissipation properties of a system, which in turn alter the measured dynamic response of the system. Plenty of information such as damage location, type and extent as well as local strain and cable tension force can be determined using ambient vibration [49,60–64]. The usage of ambient vibration as the main excitation source for vibration measurement is highly convenient since service of the civil structure needs not to be stopped. However, these methods are not very dependable in small sensor networks due their insensitivity to global damage detection. The majority of SHM research conducted over the last 30 years has attempted to identify damage in structures on a more global basis [3]. A fundamental challenge of global-based damage identification is that damage is typically a local phenomenon and may not significantly influence the lower-frequency global response of a structure that is normally measured during vibration tests. When a small number of sensors is used, global-based damage detection becomes very difficult to implement since the damage inflicted at a component level does not have enough influence in a small sensor network [65]. Moreover, when a system is then exposed to variable environmental and operational conditions such as temperature, moisture and loading that affect global vibration characteristics, the changes in dynamic response associated to these varying conditions can often mask subtler structural changes caused by damage [56,66–71].
Vibration-based damage detection methods are also affected by uncertainties in key input parameters, such as measured frequencies and mode shape data. However, when these methods are incorporated with statistical pattern recognition techniques, accuracy in structural health assessment is improved [72]. A study of the effectiveness of statistical pattern comparison and statistical model development in an unsupervised learning environment to represent the level of damage on Portage Creek Bridge in British Columbia, Canada has been performed [73]. The statistical model development approach uses an unsupervised learning technique to develop a reference model of strain variability to which subsequent data patterns are compared by means of computed residuals (R-values), while the statistical pattern comparison approach uses a data block as a reference block to which pattern from other blocks are compared.

Another example of combining vibration-based damage detection methods with unsupervised statistical pattern recognition approaches is the usage of parent and offspring finite element (FE) models calibrated with artificial neural networks to incorporate uncertainties into component and system reliability assessments. The limitations of models arise from the non-stationary nature of structural behavior induced by environmental factors. Modeling uncertainties such as boundary conditions, material properties, loads, deterioration, and damage can be included in calibrated parent and offspring models to reduce epistemic uncertainty in measurement and data post-processing [74]. A one-time initially calibrated FE model can be used to predict system reliability, but SHM data can be used to continuously calibrate a family of FE models. Artificial neural networks are used to calibrate the FE models and uncertainties in modeling, in measurement (e.g. data acquisition accuracy and sensor resolution), and in data post-processing (e.g. failure modes and assumed distributions). It has been found that by calibrating a parent model and determining offspring models that incorporate uncertainties, estimates of structural response and probability of failure become more realistic as opposed to the estimates determined using the one-time calibrated FE model [75]. This calibration method has been used to determine a distribution of load rating for a bascule bridge [74].
Time series or autoregressive models [76] have been combined with Mahalanobis distance-based outlier detection algorithms to identify changes [77,78]. Gul and Catbas [79] present a methodology where this combination is implemented and modified using random decrement functions to eliminate the stochastic effects of the input and increase separation between the reference data and the investigated data. Although statistical methods are typically useful in reducing false indications of damage [56], false negative and false positive indications in these combined methods must be reduced. In order to enhance statistical pattern recognition methods, statistical control charts and hypothesis testing modified using model spectra and residual autocorrelation together with resampling-based threshold construction methods has been proposed [80]. Ljung-Box statistic and Cosh spectral distance are the algorithms used in a study that include simulated and laboratory testing. These algorithms are found to be very conservative and more sensitive and stable than residual variance and Mahalanobis distance of coefficients.

Probabilistic or adaptive methods, such as Bayesian neural networks or extended Kalman filtering, are robust and fault tolerant and can operate with uncertain and incomplete information [81–85]. These are very attractive qualities in methods to detect damage in large civil structures, since these are often affected by loads that are not easily controlled or measured (such as traffic and wind excitation) and have small amplitude responses corrupted by noise [86,87]. Recursive Bayesian filtering is also used to identify damage and to assess structural condition and prognosis [88]. These methods, however, depend on comparing current data to previously collected data by means of adaptive parameters. More complex nodes will always offer a better data fit on the data used for learning, but over-parameterization makes poor predictions for new cases, so excessive layers must be penalized. These characteristics make these models highly complex and probabilistically dependent. Adaptive recursive least squares filtering using measured or estimated structural responses and a reasonable estimate of the input force, such as an earthquake, is used to directly identify changes in structural stiffness for the ASCE benchmark SHM problem [89]. Least mean squares algorithms are a class of adaptive filter used to mimic a desired filter by finding the filter coefficients that relate to producing the least mean squares of the error signal.
A wavelet-neural network module with a Bayesian updating scheme can be used to determine differences between measured and predicted signals [91]. The wavelet-neural network module can determine behavioral patterns of a structure [92]. This information is fed to a Bayesian updating scheme that describes the error signal between the measured signal and the signal predicted by the neural network. Ko and Ni [93] pose three reasons to prefer a neural network-based multi-stage diagnosis strategy. Neural networks can be employed for different identification purposes, so that they can fulfill monitoring objectives at different stages. They can also yield satisfactory results (identification and regional location of damage, for instance) when only modal data from a few measurement points are available. Lastly, neural network-based methods use information from forward problems at the training stage and avoid direct solution to inverse problems. The complexity of a CPS using monitoring information for actuation can be significantly reduced if such methods are used, since they are applicable to different monitoring states, provide forward problem solutions, and can effectively operate using smaller sensor networks.

As supervised learning environments collect more data to determine damage type and extent and prognosis, sensor networks must have a greater number of nodes and must be capable of handling large amounts of information. Several large-scale structures have been used as test beds to evaluate new sensing technologies and determine new areas for improvement. The Wind and Structural Health Monitoring System (WASHMS) was the largest monitoring system in the world at its time, with 800 sensors permanently installed on three cable-supported bridges [93]. Neural networks were chosen as the most favorable monitoring methodology after an exhaustive feasibility study of vibration-based damage detection methods. The study determines that, due to the low modal sensitivity of bridges to structural damage, methods that are highly tolerant to missing data, measurement noise, and structural modeling uncertainty can be applied to large bridges for vibration-based damage identification.

A very comprehensive monitoring system with several sensor networks integrated for evaluation and decision-making has been deployed at the Zhijiang Bridge [94]. This system includes an information acquisition system, a data management system, an EDM system, and an application service system. The
information acquisition system consists of several sensor networks connected to data acquisition and transmission modules by means of an anti-interference shielding line, in turn connected to a remote industrial computer via Ethernet and LAN. This data acquisition and transmission module serves as a data preprocessing and temporary storage facility. The networks used are purposed for monitoring the bridge’s working environment, including: acceleration sensors for vibration monitoring, impact force in bridge pier, earthquake response, cable tension estimation, section stress, and fatigue and crack formation monitoring; fiber grating strain sensors for anchor force monitoring at steel-concrete joint segments; optical fiber grating temperature sensors; a bridge weigh-in-motion system for vehicle load monitoring; and Global Positioning System (GPS) receivers for spatial deformation monitoring. The GPS clock is used to synchronize the two acquisition stations for stress, temperature, and vibration sensors. The data management system collects, files, inquires, stores, and manages data from the health monitoring subsystem. A data processing module in the EDM system performs statistical analyses, forecasts trends based on the collected monitoring data, and fetches key indices to report on the status of the bridge. The status evaluation module performs real-time analysis, evaluates structural status, and performs damage identification. The application service system is the user interface subsystem that allows the maintenance manager to view graphically displayed monitoring results, query monitoring points, and pre-alarming information, among other actions.

A wireless smart sensor network (WSSN) can be capable of operating with distributed data processing and triggering capabilities for power and computational efficiency for large-scale modal analysis and damage detection. The most renowned test bed to this date for its extensive and successful deployment of a WSSN is the second Jindo Bridge in Korea. The deployment consists of Imote2 smart sensor platforms, custom-designed multi-metric sensor boards SHM-A and SHM-W shown in Figure 2.4, base stations, and software provided by the Illinois Structural Health Monitoring Project (ISHMP) Services Toolsuite [95,96]. The WSSN on the Jindo Bridge is powered by solar panels and remains on sleep mode to extend its lifetime initiating monitoring upon excessive wind and vibration detected by SHM-W and SHM-A sensor boards, respectively. These functions are made possible by the service-
oriented architecture (SOA) used in the software system, which allows the usage of the same services to build different applications so that each service needs not to be adjusted for each new desired application [95]. This feature makes way to the development of different health assessment features of a structure while not expanding on complexity. Some application services in the ISHMP Toolsuite include synchronized sensing (*SyncSensing*), correlation function estimation, the Eigensystem Realization Algorithm, Stochastic Subspace Identification, Frequency Domain Decomposition, and the Stochastic Damage Locating Vector method. *SnoozeAlarm* controls the sleep-wake cycle service that allows the gateway node to gain access to leaf nodes while remaining in deep sleep mode. *ThresholdSentry* allows the usage of triggering values to awaken the necessary sensors for collection and data processing.

Figure 2.4. Multi-metric sensor boards: (a) SHM-W sensor board [96], (b) SHM-A sensor board [95].

Some other relevant applications that have been developed based on the SOA are *SHMSAutoBalance* for the SHM-S wireless strain sensor board [97], *DecentralizedDamageIdentification* [98], and *CableTensionEstimation* [99]. The SHM-S sensor combines a typical foil-type strain gauge with the friction-type magnet strain sensor, FGMH-1. The operation flow chart of the SHM-S sensor is shown in Figure 2.5. The sensor is easily and rapidly deployed, performs well in variable temperature, and is capable of overcoming the drawbacks that other strain sensors have. It records low-level ambient strain by amplifying the strain signal up to 2507 times, has better analog-to-digital converter resolution, overcomes inherent circuit noise, and it operates automatically. *DecentralizedDamageIdentification* performs output-only modal analysis using the natural excitation technique in conjunction with the Eigensystem...
Realization Algorithm. This is followed by computations for damage detection using the stochastic dynamic damage locating vector method with the maximum stress index and the average stress index. These operations are performed using a decentralized network of Imote2 nodes for better power and time efficiency. *CableTensionEstimation* uses applications provided by the ISHMP Services Toolsuite that ensure autonomous operation, sustainable energy harvesting and power consumption, and Internet remote access. Using acceleration signals, the program estimates the power spectrum to determine the natural frequencies of the cables with an automated peak-picking method, and calculates tension forces by performing linear least square fitting with the natural frequencies. This information determined within the network is then transmitted to the base station, reducing power consumption and wireless data transmission. Vibration-based cable tension force estimation sensors can also be developed from off-the-shelf commercial components. Such is the case of a cable tension force estimation system that determines tension force considering cable sag and bending stiffness [64]. Welch’s method is used to average Fourier spectra from segments of a one-time history record to remove the non-stationary qualities that short-duration signals impose. This system was validated in a laboratory setting on a reduced scale cable of the Seohae Bridge in Korea.

**Figure 2.5. Block diagram of SHM-S sensor operation** [97].
Wireless smart sensors (WSSs) present many advances that propel CPS development. The component and communication flows of typical WSSs were summarized by Lynch and Loh [100] and diagrammatically shown in Figure 2.6. On-board computation capabilities of WSSs for autonomous monitoring allow preprocessed data communication for multi-functionality in CPSs. Moreover, their low cost make the deployment of a dense array of sensors on large civil structures both economical and feasible [98]. Actuation interfaces provide a way to have on-board decision-making components in order to more effectively and quickly command actuation for controlling purposes [100]. All information collected in an SHM system can be used to enhance adaptive control and for additional controller evaluation criteria.

Figure 2.6. Components of smart wireless sensors for SHM applications [100].

The usage of low-cost equipment with on-board computing capabilities such as WSSs has allowed the deployment of highly dense sensor networks. It is desirable to have a dense array of nodes to reveal the status of a civil structure with greater resolution. However, when the network is partially destroyed due to a natural or man-made disaster, adaptive methods should then be used due to their robustness and fault tolerance. Networks such as those deployed at the Jindo and Zhijiang bridges ought to be evaluated to adapt to such a situation. The SOA provided at the WSSN of the Jindo Bridge along with the triggering capability allow a great potential robustness in face of emergency situations. Software
that incorporates adaptive methods can be developed and executed in such instances when network
density has been diminished due to an emergency situation. A decision support environment is necessary
in order to communicate alerts providing information of any anomalies detected. Alerts can be given
when part of the network is found unresponsive and response network subnets can be awakened to
provide further damage information. These alerts should also include recommendations, such as
immediate inspection, repair or activation of emergency response actions [101]. These response actions
can include the engagement of structural control systems.

2.3 Vibration-based cable tension estimation
Contrary to most civil structures, bridges experience highly variable loading conditions that result in
unique member behavior throughout their lifespan. Loading condition can be affected by cyclic vehicle
loading and natural hazards. Structural loading capacity can also be changed significantly by harsh
environmental conditions. This is particularly sensitive when the loading capacity of critical load-carrying
members is reduced. Cable-supported bridges rely on the condition of stay cables to transmit loads
effectively without compromising the safety of users. It follows then that the determination of tension
force of cables in cable-supported bridges is of special interest.

Cable loading capacity can fluctuate significantly due to changes in external loading, cable
relaxation or damage existence, whether on the member in question or in other members. Stay cables are
especially susceptible to damage. Cables are exposed cyclic changes in load pattern and wind-induced
buffeting that cause fatigue damage [102,103]. The exposure of cables to high levels of moisture can also
result in severe corrosion, reducing cable diameters up to 30% [5]. This translates into a loss in bending
stiffness of up to 76%, and hence a significant reduction in loading capacity. When loading capacity is
significantly reduced, other members may undergo excessive loading to compensate in order to complete
the load path effectively. These changes in loading condition could potentially exceed member loading
capacity, raising a safety concern. This stresses the importance of determining cable tension force
effectively.
To this end, many researchers have studied various methods to determine cable tension. One of the most common ways to directly determine cable tension is using lift-off tests [104]. In a lift-off test, a load cell and a hydraulic jack placed in series are wedged into a required space in an anchor block where the cables are secured. This method requires a space for the inclusion of a hydraulic jack in the original design of the bridge and is very costly to perform. On the other hand, other sensors such as strain gauges offer a less invasive and expensive approach to measure changes in displacement [93]. However, strain gauges can be susceptible to environmental conditions. They are also challenging to install and require direct contact with cable strands, which are not always exposed due to sheathing protection. Other strain measuring methods use Fiber Bragg grating sensors, which offer a high accuracy strain sensing option based on optical fiber that can overcome some of the limitations of strain gauges, such as the susceptibility to environmental conditions [105]. Yet, the installation of this technology remains complex and is significantly more expensive than strain gauges. Electromagnetic sensors can also be used to accurately determine cable tension, but they are even more complex to install and require a sensing coil to be turned around the unsheathed cable strand [104]. Direct measurement methods are thus labor-intensive and costly, especially when compared to vibration-based methods.

Vibration-based measurements provide modal information that can be used for multiple monitoring objectives at a much lower cost. Most vibration-based tension estimating methods use taut string theory, developed by Max Irvine in 1981 [106]. This method uses wave propagation theory to determine a simple formula using geometric and material properties to determine tension in a cable based on its natural frequencies. Several modifications to the basic formula have been added since to include bending stiffness effects, sag, varying cable mass, and end conditions [107,108]. In addition, empirical corrections and finite element-based model updating has enhanced these formulations to improve tension accuracy [109–112]. Some of these tension estimation methods have been used as algorithms in multiple developments for vibration-based cable tension estimation. Kangas et al. [63] used finite-difference model developed by Mehrabi and Tabatabai [108] to demonstrate that ambient vibration is sufficient to determine an accurate estimation of cable tension force in a cable-stayed bridge, even when the capacitive
accelerometers used collect data from the cable sheath. Cho et al. [64] used tension estimating formulas developed by Zui et al. [107] considering cable sag and flexural rigidity as embedded modules in an in-house wireless smart sensor development. Results were comparable to tension measurements determined with strain gauges. Jung et al. [49] used extended taut string formulas considering damping ratio, sag, and bending stiffness as presented by Kim and Park [111] to determine cable tension in an inclined full-scale cable in a laboratory environment with great accuracy. These applications have demonstrated the practicality of vibration-based tension estimations methods. However, further development in affordable, wireless, and expandable structural health monitoring (SHM) systems that can provide reliable tension estimation are required.

2.4 Crack detection in crack-prone areas

The USA faces a great challenge with bridge inspection for transportation safety. It has been determined that 9.1% of bridges are structurally deficient and more than half are reaching their design life [1]. In order to monitor the performance of said bridges, the Federal Highway Administration stipulates that all bridges on public roads be inspected every two years [113]. However, the reliability and frequency of inspection methods has room for improvement as structural health monitoring and nondestructive evaluation methods continue to become more informative and practical.

Some of the most failure susceptible bridge structures are steel bridges. Steel bridges account for more than 43% of substandard bridges in the USA and can deteriorate due to corrosion, increase in traffic volume, and deicing salts [114]. In addition, cyclic loading can produce local failure due to fatigue [12]. Traditionally, highway bridges use various types of connections with high stress concentration regions, which are particularly susceptible to these effects. For instance, the use of cover plates in regions of high applied moment allows sections on primary beams to be lighter while providing more flexural capacity [115,116]. However, weld defects in cover plates introduce high stress concentrations where fatigue cracks typically initiate and propagate, reducing the load capacity of the girders [117]. As another example, floor-beams are connected to main trusses by fabricating a gap that reduces the width of the
upper flange of the floor-beam right at the connection, making the beam susceptible to cracks due to out-of-plane bending and due to secondary bending moment in the plane of the floor-beam web. Other crack-prone details include diaphragms and cross-bracing connections, copied and cut-short beam ends, and stringer-to-floor beam connections. The propagation of said cracks can significantly reduce the service life of steel bridges from their original design [118]. Considering the increased traffic volume and the large percentage of steel bridges in a bridge population with high incidence of structural deficiency determined primarily by visual inspection carried out every 2 years, a system that can determine crack formation at an early stage is essential.

Many crack detecting and characterizing methods for metallic surfaces have been developed to supply this need. A common technology for flaw detection is angle beam ultrasound [119]. Another mature technology is the usage of eddy currents which reveals hidden defects at great penetration depth [120,121]. Other technologies include ultrasonic flaw detectors [122], magnetoresistance sensors [123], surface-mount piezoelectric paint sensors [124,125], probe-pump-based Brillouin sensor systems [126], coaxial cable sensors [127], fiber-optic sensors [128], acoustic emission technology [129], and large-area sensors [130,131]. Unfortunately, the use of these methods for long-term monitoring of crack patterns in larger scale civil infrastructure is time-consuming, expensive, and requires experienced operators.

Passive ultra-high frequency (UHF) radio frequency identification (RFID) antenna tags have been used in recent years to account for the costs and complexities of more established methods. RFID antennas undergo permanent changes in impedance and radiation efficiency upon changes to their surrounding electromagnetic environment, such as the presence of a crack on a metallic surface [12]. The wireless communication protocol between tag and reader is well established and reliable. Passive UHF RFID dipole tags are also very inexpensive, ranging $0.10-$0.20 per unit in mass production [132]. RFID-based sensors can also be easily assembled and installed. The wireless nature of this system along with its power independence and low cost allows it to potentially be deployed as smart dust to increase pervasiveness. Moreover, interrogation of several antenna sensors can be performed simultaneously, so
that tag arrays can provide further information on crack characteristics and pervasively monitor a large-scale structure.

Several studies have been implemented for the development of antenna sensors for crack detection with potential application to civil structures. One such project is an in-house developed wired patch antenna [9] that experimentally demonstrated the ability to determine crack length and orientation by measuring resonant frequency shifts on a conductive surface. Another successful development [8] is a dipole antenna constructed with conductive paint and a copper antenna loop that was tested on previously cracked reinforced concrete beams. In addition, a strain and crack detecting folded patch antenna was developed to monitor conductive surfaces [11].

Although all developments so far have been able to detect crack propagation in specific types of medium, none have combined 5 critical factors that any crack sensor ought to have in order to monitor steel bridge girders: (1) low cost, (2) wireless monitoring, (3) simple damage feature extraction, (4) sensitivity to crack presence on conductive surfaces, and (5) sensitivity to crack propagation on said surfaces. In order to monitor cracks propagating from areas of high stress concentration in large metallic structures using RFID technology, it is necessary to turn to lower cost, wireless options. Commercially available passive UHF RFID antenna tags can provide a solution to this problem, but have received reduced attention in comparison to in-house developments. Also, most successful developments use a vector network analyzer, which introduces an additional step of complexity in feature extraction to field users during inspection. In addition, none of the previous studies has verified sensor performance in face of a progressively propagating crack in a metallic medium with high precision measurements and ensuring small scale yielding, which is characteristic of in-service structures.
Chapter 3 Structural Cyber-physical Systems

3.1 Introduction

The state of the current transportation infrastructure has called for the development of several monitoring and mitigating strategies. The most recent report on bridge inspection practices by the Transportation Research Board states that the most common method of non-destructive testing is still visual inspection [133]. The objective of inspections is to identify the onset of damage in a timely manner to avoid structural failure. Failure is reached when damage grows, since damage itself does not imply a total loss of system functionality, but rather that the system is not operating in its optimal condition. However, visual inspections have several drawbacks. The costs and traffic interruptions incurred during visual inspection, such as the usage of platform units to identify corrosion progression in bridge girders, restrict the frequency with which these inspections can be carried out. Usually, only well-experienced trained professionals are capable of identifying damage effectively. Moreover, damage is oftentimes hidden from view so that vulnerabilities could potentially not be detected on time. Non-destructive testing and structural health monitoring (SHM) methods have been developed for objective and automated detection of damage and structural assessment.

In response to needs of performance enhancement, structural control systems have also been introduced in several types of civil structures subjected to seismic, wind and cyclic loads. High-rise buildings are particularly susceptible to wind loading and their displacement and acceleration can pose a problem to human habitation. Accelerations in medium-rise buildings and bridges due to seismic loading can cause even greater problems in terms of structural component failure. Many different structural control systems have been developed to mitigate these problems with outstanding results. Control systems can also be used to adjust loading on structural components if the required values are not met during the construction phase or after an extreme loading event.

The objective of this chapter is to introduce the concept of a structural cyber-physical system
(SCPS) as a logical solution to further improve structural control in civil structures. A cyber-physical system (CPS) is a highly complex network of embedded systems, sensor networks and actuation systems that are controlled by a computing and communication core [17]. CPSs have been introduced in many fields including medical devices, aerospace systems, transportation vehicles and factory automation. Several technological advances have contributed to the development of CPSs, including low-cost, small smart sensors, computing capacity of low-cost microcontrollers, wireless communication, abundant Internet bandwidth, and improvements in energy harvesting methods.

Research in wireless sensor networks (WSNs) is the main contributor to CPS development [134]. WSNs are mainly centered on issues concerning sensing, retrieval, communication, and coverage, while CPSs focus on the development of cross-domain intelligence for decision and actuation from multiple sensor networks. Some of the principal challenges in WSNs are also present in CPSs, including data mining, database management, and communication among multi-domain data sources. Two major features present in CPSs only include the interconnection among multiple networks and management of cross-network communication flows. Since different types of sensors are oftentimes used in CPS, data exchanges need to occur over these heterogeneous networks, posing a new challenge. The amount of collected data can also be very large, relying on data mining technologies to retrieve useful knowledge using spatial and temporal correlations.

A CPS must also have the ability of overcoming uncertainties inherently introduced by the physical system and its environment. System robustness and tolerance to component failure, both in physical and virtual domains, must be present. Synchronization across time and space between the collection, computation, communication and actuation components must be accomplished. Finally, the merging of time-based systems with event-based systems for feedback control is essential. This allows the system to become automatic so that real-time control and long-term monitoring can replace event-based damage identification systems by triggering actuation using threshold exceedance criteria.

This chapter is structured in a way to present the current state-of-the-art research in sensor and structural control technologies and how it makes way to the development of a SCPS. First, contributions
offered by structural control research are presented. Second, developments in smart sensor technology and health monitoring methods are examined. Third, current developments in CPSs for civil structure applications are reviewed. Finally, the concept of SCPS is introduced along with the research areas that must be expanded in order to attain practicability, followed by conclusions on the research review and the newly introduced concept.

3.2 Structural control research

The principal objective of structural control is to reduce structural response by determining and applying the required modifications in the structure’s dissipative capacity or by sending the appropriate signal to actuators that modify said response [135,136]. It can also be used to adjust loading on structural components if these surpass the allowed values during the construction phase. Similarly, since component capacity is typically reduced after an extreme event, structural control systems can reroute a loading pattern to reduce loading in compromised components. In order to attain these response modifications, several control systems have been engineered: passive, active, and semi-active control systems.

Passive systems are materials or devices that increase structural damping, stiffness or strength. Passive control systems are energy dissipating systems that are typically designed for a specific structure to respond to a specific type of loading. Some examples of passive control devices include friction dampers, viscoelastic (VE) dampers, viscous fluid (VF) dampers, tuned mass dampers (TMDs) [137–139], tuned liquid column dampers (TLCDs) [140,141], tuned inerter dampers [142–144], and tuned viscous mass dampers [145,146]. Friction dampers have compressive and tensile parts that slip over at a joint over a friction pad. VE dampers dissipate energy through the shear deformation induced by the relative motion between outer steel flanges and center plates that are bonded by VE layers. VF dampers consist of a piston surrounded by high viscous fluid in a casing. Tuned dampers consisting of a single damping unit can only be tuned to a single frequency, so that, while a single modal response may be reduced, response due to other modes may be increased. Response reduction to ground motion is greatest when the dominant frequency of the motion matches the frequency of a tuned damper, but is less when
the dominant frequency is different. Compared to TMDs, the frequency tuning and installation of a TLCD is much easier, fabrication is much cheaper and almost no maintenance is required [135]. A TLCD contains a body of liquid as a secondary mass that is tuned as a dynamic vibration absorber. The TLCD is highly nonlinear because of the liquid sloshing or the presence of orifices. Base isolation systems have also been developed to span passive, active and semi-active device usage [147]. One principal advantage of passive systems is that bounded-input, bounded-output structural stability is maintained, so that their inclusion in a structure does not result in an increased detrimental response regardless of the input. Passive control systems also do not require external power in order to operate, so they continue to provide damping capabilities during power outages.

Active control systems reduce response by means of controllable forcing devices that act on the structure based on data collected by sensors and calculations executed based on control laws and real-time information processing. Some active control systems include active TMDs, distributed actuators, active tendon and cable systems, active coupled building systems, and active strut control [148–150]. An active device commonly used is the active mass damper (AMD). The AMD provides a damping control force to add mechanical energy into a structure, as well as a restoring force while not occupying as much space as mechanical springs [135]. Active control devices have the great of advantage of providing a broader range of response modifications than passive devices and are also adaptable to varying loading conditions in real-time due to the sensor networks they depend on. They also tend to be smaller in size than passive devices [151]. However, their dependability on external power has made researchers turn their attention to the development of an alternate structural control system.

Semi-active damper and hybrid control systems combine the most advantageous properties of active and passive control systems. They do not require a lot of power to operate and many can do so using battery power [135,149,152,153]. Since semi-active damper systems are energy dissipating systems, they cannot destabilize the structural system in the bounded-input, bounded-output sense as an active control system can. Hybrid mass dampers, the most common control devices in full-scale civil structures, work as TMDs with the ability to actively change their tuned frequency using much less
energy than an AMD [154]. Passive components in hybrid mass dampers can also attenuate a phenomenon known as interstory response amplification, which results from active control forces that are too large for the structure to carry [155]. Semi-active TLCDs have been found to provide an additional 15-25% response reduction over a passive system [156,157]. Smart semi-active TMDs provide similar optimized results for changing dynamic characteristics using minimal energy [158–160].

Semi-active controllable fluid dampers have been developed using electrorheological (ER) or magnetorheological (MR) fluids. ER and MR fluids can change from a Newtonian fluid to a linear viscous fluid and to a semi-solid with controllable yield strength in milliseconds when exposed to an electrical or magnetic field, respectively. MR fluid dampers present several significant advantages over ER fluid dampers. ER fluid dampers have available yield stress values ranging only from 3.0 to 3.5 kPa, while MR fluid dampers attain yield stresses an order of magnitude larger. ER fluids cannot tolerate common impurities, such as water, while MR fluids exhibit no change in yield stress when exposed to the same impurities as well as impurities commonly encountered during fabrication. ER fluid dampers require high voltage and power supplies to operate, while MR fluid dampers require much less power [135,161–163]. MR fluid dampers can generate controlling forces comparable to active control systems. When a power supply is not available, an MR fluid damper can provide damping as a passive viscous fluid device. Moreover, MR fluid dampers have been shown to be highly versatile and efficient and scalable for civil engineering applications [164,165].

In a collaborative research project between the Hong Kong Polytechnic University, the Central South University of China, and the University of Illinois at Urbana-Champaign, 312 MR fluid dampers have been installed for rain and wind vibration control in the cable-stayed bridge at Dongting Lake. This is the world’s first installation of MR fluid damping technology on a bridge structure [93]. The choice of using MR fluid dampers instead of passive viscous dampers for rain and wind vibration mitigation of the cable stays is founded on two principal reasons. First, due to the varying lengths of cables, passive dampers of the same size can only achieve optimal damping of a limited number of cables while not providing sufficient damping to others. MR dampers can offer a wide range of damping capabilities.
independently of size since their damping capacities rely on the input voltage. Second, passive viscous dampers can only optimally damp a single mode of vibration. Although rain and wind induced vibrations are mostly dominated by lower frequencies, it is unclear how to predict the mode at which a certain cable will vibrate in order to determine a priori which mode must be controlled. Because of the adaptability MR dampers offer, these devices can optimally control whichever mode of vibration results as dominant after installation by changing the input voltage.

The development of versatile and powerful controlling devices such as the AMD and the MR fluid damper would not represent such a significant contribution were it not for the establishment of effective control laws. Control design evaluation test beds have been established to validate control algorithms under development. As a result, these benchmark problems also bring further information regarding the advantages and limitations of these control devices. A study presented by Jung et al. [166] evaluated a clipped-optimal control algorithm for MR fluid dampers using numerically simulated responses from seismic excitation on the ASCE first generation benchmark problem for a cable-stayed bridge in Cape Girardeau, Missouri. Three dynamic models were used and compared in the simulation: the Bingham model, the Bouc-Wen model and the modified Bouc-Wen model. The parameters needed for these models were optimized using data from experiments performed on a full-scale MR fluid damper. It was found that the control strategy applied to these MR fluid dampers reduces peak and normed responses significantly and similarly to the active control system that was evaluated for comparison. Moreover, it was found that an MR fluid damper can be dependably and manageably modeled using the more computationally tractable Bouc-Wen and modified Bouc-Wen models. In another study, the effectiveness of an MR damper to reduce peak and root-mean-square (RMS) responses on a series of experiments performed on a three-story test structure subjected to a one-dimensional ground excitation using a clipped-optimal control strategy was also evaluated [167]. In this case, the semi-active system was also found to be efficient in reducing peak and RMS responses while generating smaller control forces than the passive-on system used for comparison. A review of models of vibration control systems with hysteresis focused on the Bouc-Wen model can be found in Chang et al. [168]. The incorporation of non-
symmetrical hysteresis into the Bouc-Wen model using a genetic algorithm can be found in Kwok et al. [169]. Bahar et al. [170] present a new inverse model of MR dampers using a normalized Bouc-Wen model. Additional information on parametric modeling of the non-linear behavior of MR dampers can be found in Wang and Liao [171].

In order to standardize performance evaluation procedures for structural control algorithms, two other benchmark problems for two different control systems at the National Center for Earthquake Engineering Research have been established. These benchmark problems can be used to evaluate the effectiveness and applicability of other structural control algorithms and to assess control design issues, such as model order reduction, control-structure interaction, sensor noise, and computational delays. The first established evaluation model is based on an experimental structure of a three-story, single-bay steel building with an active mass damper [172]. A diagram of the model can be seen in Figure 3.1a. This AMD consists of a single hydraulic actuator with steel masses attached to the end of the piston rod. Natural frequencies, damping ratios, dimensions and masses per floor of the model are known and the ratios of model quantities (force, mass, time, displacement, and acceleration) to those corresponding to the prototype structure are also known. The second benchmark problem consists of a model of a three-story, tendon-controlled laboratory model shown in Figure 3.1b [173]. The structure has a hydraulic control actuator connected by a stiff steel frame to four diagonal pre-tensioned tendons at the first story. Since hydraulic actuators are open-loop unstable, a feedback control system is used to improve performance. The control system uses displacement measurements as the primary feedback signal.

The controller to be assessed for each of these benchmark problems is evaluated based on 10 criteria given in terms of RMS and peak response quantities. The RMS evaluation criteria include the controller’s ability to minimize the maximum RMS interstory drift and absolute acceleration, the required physical size of the controlling device (based on the RMS actuator displacement), the power required to operate it (based on the RMS actuator velocity), and the required magnitude of the forces to be generated (based on the RMS absolute acceleration). For the AMD benchmark problem, the peak response criteria additionally include the controller’s ability to minimize peak interstory drift, peak accelerations,
displacement and velocity of the AMD relative to the third floor, and absolute acceleration of the AMD [172]. For the active tendon system problem, the peak response criteria additionally include the controller’s ability to minimize the nondimensionalized peak interstory drifts due to the earthquake records of El Centro 1940 and Hachinohe 1968, and the ability to minimize the required control resources such that they do not exceed the control constraints of voltage, stroke, and maximum control force [173]. Benchmark problems can be further expanded to incorporate additional evaluation criteria that include efficiency of structural monitoring parameter extraction and the parameter’s contribution to improve performance of the control algorithm.

A benchmark problem for seismically excited base-isolated buildings has been used extensively to evaluate the performance of different control algorithms [174–177]. A decentralized hierarchical

Figure 3.1. NCEER benchmark problem experimental setups: (a) active mass driver system [172];
(b) active tendon system [173].

(a)  
(b)
strategy for hybrid (MR damper and passive non-linear) base isolation was evaluated using this benchmark problem [170]. Decentralization of control systems has shown to be very promising and convenient for large-scale civil structures, as they reduce feedback latency and lower demand on communication range [178,179]. Subasri et al. [180] combined active control with base isolation using a discrete direct adaptive extreme learning machine controller evaluated on this same benchmark problem. Shook et al. [181] uses the same benchmark problem to evaluate the performance of an isolation system composed of elastomeric bearings, friction-pendulum bearings, shape memory alloy wires, and MR dampers using neuro-fuzzy logic control algorithms to model the shape memory alloy and MR components. Pozo et al. [182] evaluate a static velocity-based feedback controller and a dynamic acceleration feedback controller design for robust active control of base-isolated structures. Taflanidis et al. [183] use the base-isolation benchmark problem to validate a control method whose objective is to maximize structural reliability with a stochastic simulation-based nonlinear controller design. Another evaluation of a hybrid isolation system using clipped-optimal strategy based on fuzzy control was performed [184]. Yang et al. [185] use a 5-storey model with an AMD on the roof to evaluate an algorithm of modified predictive control with direct output feedback using an offline method to find feedback gain. Johnson et al. [186] evaluated reduced-order, multiobjective optimal controllers using $l_1$ and $H_\infty$ constraints on a structural control building model benchmark developed at the University of Notre Dame. Control of response to wind excitation of a tall, slender building is also presented in a benchmark problem with an AMD system using Lyapunov functions [187].

The efficiency of a controller oftentimes depends on the available information on varying parameters. A control problem can be classified as closed-loop (feedback control) or open-loop (feedforward control). Feedback control uses the continually monitored response of the structure as input to determine the control signal to be sent back. On the other hand, the output control signal in feedforward control depends on the measured excitation. When both the system response and excitation are used to determine the control signal, the problem becomes a feedback-feedforward control system. Adaptive controllers are feedback or feedback-feedforward control mechanisms that include additional layers of
information to improve performance, including a reference model and an adjustment mechanism. Figure 3.2 diagrammatically illustrates a feedback-feedforward adaptive control strategy. The reference model is developed based on previous knowledge of the structure and represents the targeted state of the structure. The adjustment mechanism receives information from the reference model, the structure’s current state and unknown parameters, and the control signal sent to the structure. The controller calculates the control force based on the current system state and the adjustable parameters estimated by the adaptive mechanism. Many adaptive control strategies have been examined and developed for civil structures subjected to large loading.

![Diagram of Feedback-feedforward adaptive control strategy.](image)

**Figure 3.2. Feedback-feedforward adaptive control strategy.**

The key aspect of the adaptive control system is the adjustment mechanism. An evaluation on the performance of an adaptive control strategy using an adaptive mechanism based on the quadratic Lyapunov function error-estimating matrix has been performed [188]. The study proposes a systematic procedure to accelerate the convergence rate of the adjustable parameters by simultaneously magnifying the adaptation weighing coefficients in the error-estimating matrix. The performance is verified on a single-degree-of-freedom active tendon structure subjected to earthquake excitation. In another study, a time delay control algorithm for structural response control is investigated for the first time [189]. The time delay control algorithm is a simple algorithm that exhibits particular tolerance to unknown system dynamics and disturbance. Numerical and experimental validations of the algorithm for an active mass damper (AMD) system for vibration suppression of a building structure under wind loading are
performed. It is found that the strategy is effective to reduce acceleration response comparably to the LQG control algorithm for an AMD system [190]. The researchers responsible for this study successfully address potential practical issues such as determining the allowable control gain range and reducing time delay. Adaptive control algorithms have also been developed for nonlinear response models. In Ikhouane et al. [191], an adaptive controller is designed to obtain performance bounds and corroborate the closed-loop stability of a Bouc-Wen model. The nonlinear response is found to be uniformly bounded and the efficiency of the controller to reduce response to external excitation is verified via numerical simulations. These studies represent significant contributions in adaptive control algorithms applicable to active and semi-active control devices that overcome challenges presented by unknown system parameters.

The deployment of control systems in civil structures can quickly escalate in complexity and cost if tethered sensor networks are used. The development of wireless sensors to command actuators represents a significant contribution to address this issue. Yet, bandwidth and range limitations in wireless communication channels can only be overcome by using decentralized networks. Partially decentralized linear quadratic regulation control schemes using redundant state estimation to minimize data communication among sensors have been developed [36,192]. The strategy is validated using numerical simulations and laboratory experiments of a seismically excited six-story building with semi-active control devices. By embedding stead-state Kalman estimators, the wireless sensors are shown to be capable of collecting output and supplying actuator commands. The resulting state estimates are compared to local measured data. Feedback control forces are computed using these local estimates when the error between measured and estimated state data is small. When the error is larger than a specified threshold value, the measured values are wirelessly transmitted to the network so that the other sensors can update their own estimates. This minimizes wireless communication, consequently saving power and reducing data loss.

The recent advances in structural control presented in this section represent crucial steps towards the intelligent integration of control and health monitoring. Adaptive algorithms overcome the drawbacks imposed by the lack of structural parameter information, which is ideal in instances of partially downed
networks. However, if a network is complete or partially present, improved parameter estimations can be obtained through an evaluation and decision-making (EDM) system. Depending on the promptness required for response, either rapid on-line or off-line evaluation could be performed to increase robustness in the control strategy. Decentralized wireless networks used in structural control can also incorporate SHM capabilities to enhance robustness in control algorithms. In addition, with the development of semi-active damping devices, control forces comparable to those provided by active devices can be attained while preserving power. In case of a power outage, a semi-active device becomes a passive device, which, in spite of not necessarily being optimally tuned, still improves response while guaranteeing bound-input, bound-output structural stability.

3.3 Structural health monitoring and smart sensing technology

SHM can be carried out as a long-term strategy where information on the structure’s ability to continue operating is evaluated in light of inevitable aging and damage accumulation. The process involves the observation of the structure over time using periodically spaced measurements, extraction of damage sensitive features from these measurements and a statistical analysis of these features to determine the current state of the system [6]. Rapid condition screening is oftentimes used to provide real-time and reliable information about system performance during and after an extreme event [3].

The statistical analysis of extracted damage sensitive features helps discriminate between features from the damaged and undamaged states of the structure in question [7]. Statistical models help answer questions regarding the existence, location, type, and extent of damage. They can also offer information on the structure’s prognosis [50–55]. When data are available from both the undamaged and damaged states, the statistical pattern recognition algorithm falls into a general classification known as supervised learning. Unsupervised learning occurs when the algorithms are applied to data that does not contain damaged samples [56]. Unsupervised learning environments can only provide information on existence and location of damage. The main advantage of supervised learning is that damage type and extent and prognosis can be determined due the availability of correlated measured features [57–59].
Vibration-based monitoring methods allow a versatility that other nondestructive evaluation methods may not. The basic premise of most vibration-based damage detection methods is that damage will alter the stiffness, mass, or energy dissipation properties of a system, which in turn alter the measured dynamic response of the system. Plenty of information such as damage location, type and extent as well as local strain and cable tension force can be determined using ambient vibration \([49,60–64]\). The usage of ambient vibration as the main excitation source for vibration measurement is highly convenient since service of the civil structure needs not to be stopped. However, these methods are not very dependable in small sensor networks due their insensitivity to global damage detection. The majority of SHM research conducted over the last 30 years has attempted to identify damage in structures on a more global basis \([3]\). A fundamental challenge of global-based damage identification is that damage is typically a local phenomenon and may not significantly influence the lower-frequency global response of a structure that is normally measured during vibration tests. When a small number of sensors is used, global-based damage detection becomes very difficult to implement since the damage inflicted at a component level does not have enough influence in a small sensor network \([65]\). Moreover, when a system is then exposed to variable environmental and operational conditions such as temperature, moisture and loading that affect global vibration characteristics, the changes in dynamic response associated to these varying conditions can often mask subtler structural changes caused by damage \([56,66–71]\).

Vibration-based damage detection methods are also affected by uncertainties in key input parameters, such as measured frequencies and mode shape data. However, when these methods are incorporated with statistical pattern recognition techniques, accuracy in structural health assessment is improved \([72]\). A study of the effectiveness of statistical pattern comparison and statistical model development in an unsupervised learning environment to represent the level of damage on Portage Creek Bridge in British Columbia, Canada has been performed \([73]\). The statistical model development approach uses an unsupervised learning technique to develop a reference model of strain variability to which subsequent data patterns are compared by means of computed residuals (R-values), while the
statistical pattern comparison approach uses a data block as a reference block to which pattern from other blocks are compared.

Another example of combining vibration-based damage detection methods with unsupervised statistical pattern recognition approaches is the usage of parent and offspring finite element (FE) models calibrated with artificial neural networks to incorporate uncertainties into component and system reliability assessments. The limitations of models arise from the non-stationary nature of structural behavior induced by environmental factors. Modeling uncertainties such as boundary conditions, material properties, loads, deterioration, and damage can be included in calibrated parent and offspring models to reduce epistemic uncertainty in measurement and data post-processing [74]. A one-time initially calibrated FE model can be used to predict system reliability, but SHM data can be used to continuously calibrate a family of FE models. Artificial neural networks are used to calibrate the FE models and uncertainties in modeling, in measurement (e.g. data acquisition accuracy and sensor resolution), and in data post-processing (e.g. failure modes and assumed distributions). It has been found that by calibrating a parent model and determining offspring models that incorporate uncertainties, estimates of structural response and probability of failure become more realistic as opposed to the estimates determined using the one-time calibrated FE model [75]. This calibration method has been used to determine a distribution of load rating for a bascule bridge [74].

Time series or autoregressive models [76] have been combined with Mahalanobis distance-based outlier detection algorithms to identify changes [77,78]. Gul and Catbas [79] present a methodology where this combination is implemented and modified using random decrement functions to eliminate the stochastic effects of the input and increase separation between the reference data and the investigated data. Although statistical methods are typically useful in reducing false indications of damage [56], false negative and false positive indications in these combined methods must be reduced. In order to enhance statistical pattern recognition methods, statistical control charts and hypothesis testing modified using model spectra and residual autocorrelation together with resampling-based threshold construction methods has been proposed [80]. Ljung-Box statistic and Cosh spectral distance are the algorithms used...
in a study that include simulated and laboratory testing. These algorithms are found to be very conservative and more sensitive and stable than residual variance and Mahalanobis distance of coefficients.

Probabilistic or adaptive methods, such as Bayesian neural networks or extended Kalman filtering, are robust and fault tolerant and can operate with uncertain and incomplete information [81–85]. These are very attractive qualities in methods to detect damage in large civil structures, since these are often affected by loads that are not easily controlled or measured (such as traffic and wind excitation) and have small amplitude responses corrupted by noise [86,87]. Recursive Bayesian filtering is also used to identify damage and to assess structural condition and prognosis [88]. These methods, however, depend on comparing current data to previously collected data by means of adaptive parameters. More complex nodes will always offer a better data fit on the data used for learning, but over-parameterization makes poor predictions for new cases, so excessive layers must be penalized. These characteristics make these models highly complex and probabilistically dependent. Adaptive recursive least squares filtering using measured or estimated structural responses and a reasonable estimate of the input force, such as an earthquake, is used to directly identify changes in structural stiffness for the ASCE benchmark SHM problem [89]. Least mean squares algorithms are a class of adaptive filter used to mimic a desired filter by finding the filter coefficients that relate to producing the least mean squares of the error signal (difference between the desired and the actual signal) [90]. A wavelet-neural network module with a Bayesian updating scheme can be used to determine differences between measured and predicted signals [91]. The wavelet-neural network module can determine behavioral patterns of a structure [92]. This information is fed to a Bayesian updating scheme that describes the error signal between the measured signal and the signal predicted by the neural network. Ko and Ni [93] pose three reasons to prefer a neural network-based multi-stage diagnosis strategy. Neural networks can be employed for different identification purposes, so that they can fulfill monitoring objectives at different stages. They can also yield satisfactory results (identification and regional location of damage, for instance) when only modal data from a few measurement points are available. Lastly, neural network-based methods use information
from forward problems at the training stage and avoid direct solution to inverse problems. The complexity of a CPS using monitoring information for actuation can be significantly reduced if such methods are used, since they are applicable to different monitoring states, provide forward problem solutions, and can effectively operate using smaller sensor networks.

As supervised learning environments collect more data to determine damage type and extent and prognosis, sensor networks must have a greater number of nodes and must be capable of handling large amounts of information. Several large-scale structures have been used as test beds to evaluate new sensing technologies and determine new areas for improvement. The Wind and Structural Health Monitoring System (WASHMS) was the largest monitoring system in the world at its time, with 800 sensors permanently installed on three cable-supported bridges [93]. Neural networks were chosen as the most favorable monitoring methodology after an exhaustive feasibility study of vibration-based damage detection methods. The study determines that, due to the low modal sensitivity of bridges to structural damage, methods that are highly tolerant to missing data, measurement noise, and structural modeling uncertainty can be applied to large bridges for vibration-based damage identification.

A very comprehensive monitoring system with several sensor networks integrated for evaluation and decision-making has been deployed at the Zhijiang Bridge [94]. This system includes an information acquisition system, a data management system, an EDM system, and an application service system. The information acquisition system consists of several sensor networks connected to data acquisition and transmission modules by means of an anti-interference shielding line, in turn connected to a remote industrial computer via Ethernet and LAN. This data acquisition and transmission module serves as a data preprocessing and temporary storage facility. The networks used are purposed for monitoring the bridge’s working environment, including: acceleration sensors for vibration monitoring, impact force in bridge pier, earthquake response, cable tension estimation, section stress, and fatigue and crack formation monitoring; fiber grating strain sensors for anchor force monitoring at steel-concrete joint segments; optical fiber grating temperature sensors; a bridge weigh-in-motion system for vehicle load monitoring; and Global Positioning System (GPS) receivers for spatial deformation monitoring. The GPS clock is
used to synchronize the two acquisition stations for stress, temperature, and vibration sensors. The data management system collects, files, inquires, stores, and manages data from the health monitoring subsystem. A data processing module in the EDM system performs statistical analyses, forecasts trends based on the collected monitoring data, and fetches key indices to report on the status of the bridge. The status evaluation module performs real-time analysis, evaluates structural status, and performs damage identification. The application service system is the user interface subsystem that allows the maintenance manager to view graphically displayed monitoring results, query monitoring points, and pre-alarming information, among other actions.

A wireless smart sensor network (WSSN) can be capable of operating with distributed data processing and triggering capabilities for power and computational efficiency for large-scale modal analysis and damage detection. The most renowned test bed to this date for its extensive and successful deployment of a WSSN is the second Jindo Bridge in Korea. The deployment consists of Imote2 smart sensor platforms, custom-designed multi-metric sensor boards SHM-A and SHM-W shown in Figure 3.3, base stations, and software provided by the Illinois Structural Health Monitoring Project (ISHMP) Services Toolsuite [95,96]. The WSSN on the Jindo Bridge is powered by solar panels and remains on sleep mode to extend its lifetime initiating monitoring upon excessive wind and vibration detected by SHM-W and SHM-A sensor boards, respectively. These functions are made possible by the service-oriented architecture (SOA) used in the software system, which allows the usage of the same services to build different applications so that each service needs not to be adjusted for each new desired application [95]. This feature makes way to the development of different health assessment features of a structure while not expanding on complexity. Some application services in the ISHMP Toolsuite include synchronized sensing (SyncSensing), correlation function estimation, the Eigensystem Realization Algorithm, Stochastic Subspace Identification, Frequency Domain Decomposition, and the Stochastic Damage Locating Vector method. SnoozeAlarm controls the sleep-wake cycle service that allows the gateway node to gain access to leaf nodes while remaining in deep sleep mode. ThresholdSentry allows the usage of triggering values to awaken the necessary sensors for collection and data processing.
Some other relevant applications that have been developed based on the SOA are SHMSAutoBalance for the SHM-S wireless strain sensor board [97], DecentralizedDamageIdentification [98], and CableTensionEstimation [99]. The SHM-S sensor combines a typical foil-type strain gauge with the friction-type magnet strain sensor, FGMH-1. The operation flow chart of the SHM-S sensor is shown in Figure 3.4. The sensor is easily and rapidly deployed, performs well in variable temperature, and is capable of overcoming the drawbacks that other strain sensors have. It records low-level ambient strain by amplifying the strain signal up to 2507 times, has better analog-to-digital converter resolution, overcomes inherent circuit noise, and it operates automatically. DecentralizedDamageIdentification performs output-only modal analysis using the natural excitation technique in conjunction with the Eigensystem Realization Algorithm. This is followed by computations for damage detection using the stochastic dynamic damage locating vector method with the maximum stress index and the average stress index. These operations are performed using a decentralized network of Imote2 nodes for better power and time efficiency. CableTensionEstimation uses applications provided by the ISHMP Services Toolsuite that ensure autonomous operation, sustainable energy harvesting and power consumption, and Internet remote access. Using acceleration signals, the program estimates the power spectrum to determine the natural frequencies of the cables with an automated peak-picking method, and calculates tension forces by performing linear least square fitting with the natural frequencies. This information determined within the network is then transmitted to the base station, reducing power consumption and wireless data.
transmission. Vibration-based cable tension force estimation sensors can also be developed from off-the-shelf commercial components. Such is the case of a cable tension force estimation system that determines tension force considering cable sag and bending stiffness [64]. Welch’s method is used to average Fourier spectra from segments of a one-time history record to remove the non-stationary qualities that short-duration signals impose. This system was validated in a laboratory setting on a reduced scale cable of the Seohae Bridge in Korea.

**Figure 3.4. Block diagram of SHM-S sensor operation** [97].

Wireless smart sensors (WSSs) present many advances that propel CPS development. The component and communication flows of typical WSSs were summarized by Lynch and Loh [100] and diagrammatically shown in Figure 3.5. On-board computation capabilities of WSSs for autonomous monitoring allow preprocessed data communication for multi-functionality in CPSs. Moreover, their low cost make the deployment of a dense array of sensors on large civil structures both economical and feasible [98]. Actuation interfaces provide a way to have on-board decision-making components in order to more effectively and quickly command actuation for controlling purposes [100]. All information
collected in an SHM system can be used to enhance adaptive control and for additional controller evaluation criteria.

Figure 3.5. Components of smart wireless sensors for SHM applications [100].

The usage of low-cost equipment with on-board computing capabilities such as WSSs has allowed the deployment of highly dense sensor networks. It is desirable to have a dense array of nodes to reveal the status of a civil structure with greater resolution. However, when the network is partially destroyed due to a natural or man-made disaster, adaptive methods should then be used due to their robustness and fault tolerance. Networks such as those deployed at the Jindo and Zhijiang bridges ought to be evaluated to adapt to such a situation. The SOA provided at the WSSN of the Jindo Bridge along with the triggering capability allow a great potential robustness in face of emergency situations. Software that incorporates adaptive methods can be developed and executed in such instances when network density has been diminished due to an emergency situation. A decision support environment is necessary in order to communicate alerts providing information of any anomalies detected. Alerts can be given when part of the network is found unresponsive and response network subnets can be awakened to provide further damage information. These alerts should also include recommendations, such as immediate inspection, repair or activation of emergency response actions [101]. These response actions can include the engagement of structural control systems.
3.4 Current state of structural cyber-physical systems

A CPS is a confluence of embedded systems, real-time systems, distributed sensor systems and controls whose operations are monitored, coordinated, controlled and integrated by a computing and communication core [17]. A CPS bridges the virtual world of computing and communications with the continuous physical world using interconnected processing elements in wired or wireless networks connecting smart sensors to actuators. Some developments that have contributed to the implementation of CPSs are the availability of low-cost, small smart sensors; the computing capacity of low-cost, reduced-size microcontrollers; wireless communication; abundant internet bandwidth; and improvements in energy harvesting methods. Challenges that have been identified in any general CPS application include: the ability of computing components to overcome uncertainties inherently introduced by the physical system and its environment; synchronization across time and space between collection, computation, communication, and actuation components; robustness and tolerance of the system to component failure, either in the physical or virtual domain; development of smaller and more powerful actuators; and merging of time-based systems with event-based systems for feedback control. CPSs have been applied in medical devices, aerospace systems, transportation vehicles, defense systems, robotic systems, process control, factory automation, emergency management, and environmental control [17,18,27–31,19–26].

Several studies present CPS design approaches to civil engineering applications in SHM, structural control, and combined situations. Real-time hybrid testing presents a challenging CPS where physical and computational components must be perfectly synchronized at run-time in order to achieve reliable results. Huang et al. [32] evaluate the efficiency of a middleware architecture to maintain predictable timing between all physical and virtual components. Another study presents the use of CPSs to monitoring temporary structures for the improvement of safety in the construction industry [33]. One CPS design approach has been developed to satisfy the health monitoring (i.e. physical) requirements and the constraints imposed by a WSN (i.e. virtual component) [34]. The limitations to a centralized network architecture are apparent: data can only be collected from a reduced number of nodes in a reasonable time frame, which results in the detection of only the most severe damage. This means that a timely detection
of structural failure resulting from extreme events, such as an earthquake or an explosion, is not possible [35]. Since WSNs incur in high-energy consumption and long delays when sensors are used as simple data collection devices, a multi-level computing architecture is proposed to selectively activate additional sensors only in the damaged regions, allowing much of the network to remain asleep. This is accomplished by using a hierarchical decentralized system consisting of grouping nodes into clusters. Cluster members collect raw data from their accelerometers and transform their data into the frequency domain through Fast Fourier Transforms and power spectrum analysis. This information is communicated to the cluster head motes, where cross-spectral density and singular value decomposition is carried out to extract the structure’s mode shape vector and communicate it to the base station. The current flexibility matrix is calculated at this level and used to determine the existence and location of damage. The approach is tested using the Intel Imote2 platform with TinyOS software on a cantilever beam with single damage and a simulated truss with multiple damage locations and intensities. Although this study addresses the issues presented by the limitations of WSSs as physical constraints, it does not include any command computed in a virtual space to affect a physical component.

A type of CPS of great interest is the wireless structural control (WSC) system. This type of CPS uses a feedback control loop to influence the dynamic response of structures using sensor data collected through WSNs. As such, WSC systems play a crucial role in protecting civil infrastructure in the event of earthquakes and other disasters. Unfortunately, since WSC systems are so expensive and time-consuming to deploy, most research performed on them has been on laboratory-scale structures. Such is the case of the WSC system tested by Swartz and Lynch [36], where embedded steady-state Kalman estimators are used to minimize wireless communication in a seismically excited laboratory-scale six-story building. Because of this testing limitation, the delays and data losses that would be expected to occur in wireless networks deployed on large civil structures is not captured and so has not been exhaustively addressed. This problem has been partially resolved by developing a Wireless Cyber-Physical Simulator (WCPS) that combines realistic simulations of WSNs and structures [37]. WCPS integrates Simulink to represent structural system dynamics and the controller with TOSSIM to simulate the WSN based on realistic
wireless link models. The interfaces between the Simulink model and TOSSIM are the Interfacing Block and the Data Block, two MATLAB embedded functions in Simulink (see Figure 3.6). The WCPS has been used to develop a WSC benchmark problem for an active mass driver [38]. This benchmark problem provides a method to evaluate wireless control design issues such as network-induced delay, data loss, available sensor measurements, and measurement noise.

Figure 3.6. Component architecture of the Wireless Cyber-Physical Simulator [37].

In a strict sense, structural control strategies are CPSs since physical information is collected and used to determine physical actions in a cyber realm. However, the complexity and cross-domain communication between several networks for intelligent decisions that characterizes a CPS is not present in these systems. The delays incorporated into a more complex wireless, networked control systems are being studied and tended to with improvements to communication protocols [39]. In order to improve stability and performance (i.e. minimize packet loss and time-varying delays), a passivity-based architecture for a robotic system has been designed and tested [40]. The use of wireless networks for control represents a significant step towards the incorporation of global and component status into the controlling algorithm.

Some studies have commenced integration between SHM and controlling systems in civil structures. The Guangzhou New TV Tower (GNTVT), also known as the Canton Tower, is the most
heavily instrumented super tall structure in the world. Its complicated SHM system was designed and implemented by the Hong Kong Polytechnic University for in-construction and in-service monitoring. The integration of in-construction and in-service monitoring strategies allows the establishment of a dynamically calibrated baseline model, a model that updates modal information at various stages of construction until completion [41]. This type of baseline model eases computational effort when substructure techniques are used. The GNTVT has inspired several investigations and developments that include new monitoring frameworks to improve wireless communication distance [42], improvements on sensor placement [43], evaluation of vibration-based SHM and damage detection methods [44,45], methods to eliminate noise from vibration responses [46], deformation monitoring [47], and modal parameter identification and updating for high-rise structures [48].

The SHM system consists of six modules: a sensory system, a data acquisition and transmission system, a data processing and control system, a data management system, a structural health evaluation system, and an inspection and maintenance system (see Figure 3.7). Sensors collect data on loading sources, structural response and environmental conditions. The on-line condition evaluation system compares measurement data with design values, FEM analysis results, and predetermined patterns and thresholds for quick assessment. The off-line condition evaluation system consists of damage diagnostic and prognostic algorithms for a more detailed health and safety assessment.

Information from the SHM system is used to verify the effectiveness of a wind vibration control system. The hybrid control system consists of two tuned mass dampers coupled with two active mass dampers and two tuned mass dampers suspended at different heights. The control system is activated by signals from anemometers and a seismograph. Ad hoc transducers provide feedback to the vibration control algorithm. These ad hoc signals are also transmitted to the monitoring center for comparison with structural response signals to detect possible faults in the ad hoc transducers. This technique of redundancy and cross-domain networking is a step closer to the integration of monitoring and actuation networks, which characterizes a CPS.
Another study that integrates SHM and structural control systems proposes an energy harvesting, cable tension-estimating and vibration-controlling strategy (see Figure 3.8) [49]. In order to supply energy to wireless sensors and an MR damper, an electromagnetic induction device is used. It was found that the electromagnetic induction device generated sufficient energy to operate an Imote2 wireless sensor node twice per day for a month. This translates into enough power to operate 45 Imote2 sensors for a one-time sensing. Free vibration tests were also performed to evaluate cable tension estimation and vibration controlling capabilities. It was found that electromagnetic field signals provided similar power spectral information as acceleration signals to estimate cable tension with 2.5% error. The MR damper also provides damping 20% larger than a passive optimally tuned device.
Although this multi-functional system and the GNTVT systems are irrefutably combining SHM and structural control functionalities on the same structures, they do not address several issues related to the development of an SCPS. The interconnectivity required between SHM and structural control sensing networks and virtual components that define an SCPS has not been addressed in any study to this date. Information sharing between monitoring and control systems is necessary to continually validate effectiveness of both systems and to provide additional health assessment and control criteria for smarter decisions. Full-scale implementations of WSC systems still need to be performed. Benchmark problems that incorporate SHM evaluation criteria into controlling algorithms need to be developed. The following section will define what an SCPS should consist of, what aspects have already been addressed and which problems still need to be researched.

### 3.5 Proposed paradigm for structural cyber-physical systems

An SCPS is an autonomous, comprehensive system targeted to improve structural performance and maintenance integrated with an alert system for public safety. An SCPS consists of two main overlapping systems: a control system and a monitoring system. Figure 3.9 shows the communication sequences to be expected in an SHM system, a structural control system, and in an SCPS. All three systems consist of virtual (circular) and physical (rectangular) components that depend on complete and stable communication to operate efficiently.
Figure 3.9. Communication of components in a: (a) structural control system, (b) structural health monitoring system, (c) structural cyber-physical system.
Communication in an SCPS begins when a structure encounters an excitation. This excitation may be seismic, wind or impact in case of immediate structural response control, or it may be ambient vibration for purposes of maintenance or a post-event assessment and mitigation strategy. The excitation may or may not be recorded by a sensing network, represented by the dashed communication arrow between the excitation and the sensing network in Figure 3.9c, depending on the nature of the monitoring and control algorithms employed in the specific SCPS. Control algorithms that necessitate to record excitation make part of feedforward control strategies. Feedback control and output-only SHM strategies can be performed without excitation information. However, structural response must always be measured by sensing networks. This is so because the SCPS contains integrated SHM and alert systems, which fully rely on structural response for their operations.

Sensing networks are both physical and virtual components as long as they consist of the physical sensing element interfaced to at least one digitizing feature, such as an integrated analog-digital converter, memory, or on-board computational ability. Since sensing devices are the last physical components in the SCPS workflow before virtual components begin to take part, networks that combine physical and virtual components are preferred in an SCPS because they can facilitate the subsequent communication flows without requiring human intervention to proceed. WSNs are an example of sensing networks that interface physical and virtual features. In SCPSs, the selection of a sensing network must also be carefully made based on the hazards the structure is anticipated to face. For instance, networks in areas of high seismic risk must be capable of triggering upon reaching a threshold in acceleration and have a large measurement range and sampling frequency to capture significant time-domain and frequency-domain information about the excitation and structural response without saturation or aliasing.

If WSSNs are used, data can be pre-processed to filter out unnecessary noise and find abnormal structural behavior. Useful data can be extracted on-board and then sent to an EDM system and to a controller. Nodes in WSSNs can share information with each other to increase network reliability and operability. This data can be sent to an EDM system for damage identification, general structural
condition assessment, and prognosis. Alerts can be transmitted to stakeholders or the public via Internet, radio or cellular towers so that timely safety actions can take place.

The data sent to the EDM system can be statistically analyzed to enhance the controlling algorithm with additional performance criteria. In order to allow autonomy in an SCPS, the controller and the EDM system must be able to operate independently, but allowing data flows from the EDM system to the controller to serve as enhancements to performance criteria. A reference model in both systems allows redundancy by comparing and adjusting decision-critical information using probabilistic and adaptive methods, resulting in smarter decisions. The usage of active or semi-active control devices is necessary in an SCPS in order to implement adaptive control strategies. The adjustment mechanism in a feedback-feedforward adaptive control system allows the inclusion of updated information on the structure resulting from the condition assessment performed at the EDM system level. Information such as current loading condition and capacity is critical to improve resiliency and safety of structural control systems, and it can only be provided by SHM assessments. For example, tension estimation and damage identification are fundamental monitoring information in bridge cable structures. In order to implement an automatic tension control strategy, the output control tension force will depend on loading changes and damage information to update the reference model (refer to Figure 3.2). The controller can then operate physical actuators for structural response control more safely and accurately.

An SCPS such as the one described above has many requirements at each step of operation. The technologies needed to commence SCPS development can be found in much of the SHM and structural control research presented thus far. Due to the large number of sensors that are required for a dependable health monitoring scheme on a large civil structure, control and monitoring systems that require no excitation recording will likely be favored to reduce complexity and power consumption. Output-only damage detecting and probabilistic monitoring methods are very useful in these situations, as well as robust and fault-tolerant adaptive control systems. As adaptive control systems have been tested for both linear and nonlinear response models, these systems have been shown to be useful to control structures that have reached local plasticity, as is common during seismic events.
The applicability of adaptive control algorithms to semi-active control devices presents a great opportunity to combine the robustness and fault tolerance of an algorithm to the versatility and economic advantages that semi-active control devices have. MR fluid dampers, for instance, have been shown to be scalable for civil engineering applications and provide damping forces comparable to those offered by active control devices. These control forces can be attained in milliseconds by changing input voltage. MR fluid dampers do not require a lot of power to operate and behave as a passive viscous fluid damper during power outs, which can often be encountered in an emergency situation. MR fluid dampers can also be operated on battery power. Since these devices are energy dissipating mechanisms, structural stability is ensured if a command malfunction was to occur. Moreover, MR fluids can achieve great yield stresses in spite of temperature variation and impurities. Research involving MR fluid dampers has been greatly facilitated by the development of computationally tractable models for simulation, such as the Bouc-Wen and modified Bouc-Wen models.

Low-cost WSSs for on-board processing can be used to deploy large networks and avoid data losses while saving power and time during communication. Decentralization strategies can be used to overcome bandwidth and range limitations. The Imote2 platform has been extensively used in these research topics and their SOA has made way for further programming possibilities of new algorithms for monitoring strategies. This feature can be used for the development of algorithms that allow cross-communication between control and monitoring systems. The research performed in WSC systems provides the first step toward this cross-communication requirement. Research in time delay control algorithms can enhance WSC systems to expand their networks for larger structures.

Additional cross-domain interactions between the monitoring and control systems still need to be addressed. Queuing and scheduling of networks to prevent interference issues need to be investigated. Efficient filtering and feature extraction methods for the controller and evaluation and decision-making levels need to be defined and improved. Communication between the EDM system and the controller needs to be addressed for compatibility issues and possible time delays. Merging of time-based systems with event-based systems is necessary to simultaneously perform efficient real-time control and long-term
monitoring, as well as to perform event-based damage identification and make way for threshold triggering systems. Control criteria need to be developed to include monitoring results such as damage state, deformation, stresses, strains and internal forces at the component and global level. Time delays in control algorithms need to be further investigated within expanded benchmark problems that simultaneously evaluate monitoring and control systems. Some adaptive control algorithms, such as the clipped-optimal control strategy and the time delay control algorithm, have been tested on competent control devices. Incorporation of monitoring data into the ASCE first generation benchmark problem for the cable-stayed bridge in Cape Girardeau is recommendable. Finally, as greater power demands are expected due to the increased complexity that an SCPS entails, new power harvesting methods will consequently need to be developed as critical system components.

3.6 Conclusions
An SCPS offers a viable and convenient option for increased safety in buildings and transportation infrastructure. An SCPS incorporates the benefits of an SHM system with structural control systems by including monitoring data to enhance controlling actions. During and after disastrous events, sensing networks may result affected, partially losing response or excitation measurements. Adaptive methods offer a solution to this lack of information due to the partially downed sensing networks. Probabilistic and statistical pattern recognition methods are also pertinent for increased robustness due to their fault tolerance.

Semi-active control devices are the most appropriate type of actuator for an SCPS. Several devices have been successfully scaled and used in civil engineering applications. MR fluid dampers, for instance, have been found to be insensitive to variations in environmental conditions. They also possess other advantages that qualify them for emergency situations, such as low power consumption and battery operability, continued passive damping capabilities in the event of a power outage, and manageable numerical modeling for realistic simulations. The use of WSSs in WSC systems is desirable for large systems that need to reduce cost and complexity. WSSs allow autonomous monitoring with actuation.
possibilities. Triggering options and network decentralization strategies are available for power and computational efficiencies. The SOA in the Imote2 platform allows programming expansion for multi-functionality. These multiple functions can include filtering, cleansing, scheduling, triggering and partial processing. This monitoring data can be further processed at an EDM system to then be used for the enhancement of adaptive control strategies.

Cross-domain interactions and communication protocols between control and monitoring systems need to be addressed. Relevant control criteria originated from monitoring data needs to be determined. Benchmark control problems must be expanded to include structural status as evaluation criteria. Monitoring and control test beds with deployed sensing networks such as the second Jindo Bridge, the Zhijiang Bridge, the GNTVT and the WASHMS exist and may need relatively minor modifications to test the SCPSs. Further power harvesting methods must be investigated to supply the increased demand directly proportional to the complexity of an SCPS.
Chapter 4 Experimental Validation and Automation of a Vibration-based Damage Detection Method

4.1 Introduction
The condition of transportation infrastructure in the United States is detrimental. An average of 188 million trips are made over structurally deficient bridges every day [1]. Thus, inspection, maintenance and repair of bridges are of paramount importance for the safety of the population. Significant effort in the development of more effective inspection and monitoring systems must be placed in order to catch up with the great number of damaged bridges in the country. Inspectors particularly face a great challenge in evaluating the condition of critical load-carrying elements. However, visual inspections are still the most common method of assessing bridge structural integrity [193] and much can be missed during a visual inspection.

For instance, steel girders in highway bridges are located under the deck and endure extreme corrosion due to environmental exposure and moisture accumulation at expansion joints. The situation in cable-stayed bridges is often even more complex for visual inspection, since main tension-resisting elements are often hidden under protective sheathing. The presence of steel protective pipes also restricts the number of nondestructive testing (NDT) methods that can be used to identify damage. Moreover, the most difficult region to evaluate is the critical anchorage zone since it is hidden from view and most NDT methods cannot be performed in this region [193]. This makes vibration-based methods more advantageous for these purposes. As such, a dependable damage detection method that can locate damage without yielding false positive or false negative locations is needed. Quantifying the extent of damage using sensitive parameters must also be an integral part of such a method.

The study presented in this chapter investigates the effectiveness of a modified flexibility-based damage identification method to detect, locate and quantify damage in a shear building and in cable structures near the anchor zone. The eigenparameter decomposition (ED) of structural flexibility change
method uses the modal flexibility matrix to determine damage in a structure [16]. It has been observed that one of the greatest advantages of using the modal flexibility matrix instead of the stiffness matrix to detect damage is that the modal flexibility matrix can be accurately determined using only a few of the lower frequency modes (which are less prone to measurement errors produced by noise) and that it is very sensitive to damage [15]. However, this method has been shown in the past to be able to locate and calculate damage extent in a purely analytical framework [16].

The work presented in this chapter consists of validating the ED method via a simulated cable structure and on a reduced laboratory-scale shear building in an experimental framework. The modal flexibility matrices for the structures used to validate the ED method were determined using the structure’s natural frequencies and mass-normalized modes. The theory of the method is explained and the effects of damage severity and cable damping are evaluated. Three steps in the method that represent a challenge in the automatic execution of the algorithm are also enhanced to reduce user interaction to a necessary minimum. The performance in damage location is also compared to that of a damage index method. In contrast to the ED method, this damage detection method uses only the mode shapes to determine damage indicators per mode, damage indicators considering all modes or damage indicators considering a few modes at once. Although it does not take advantage of the sensitivity of the flexibility matrix to damage nor does it determine the extent of damage, it is an extensively validated and reliable method to determine location of damage in civil structures [194,195]. For this reason it has been chosen to evaluate the effectiveness of the ED method in locating damage. The following sections describe the theory behind the modal flexibility matrix, the ED method and the damage index method.

4.2 Theoretical background
The ED method uses the flexibility matrix to determine the damage in a structure. It has been observed that one of the greatest advantages of using the modal flexibility matrix instead of the stiffness matrix to detect damage is that the modal flexibility matrix can be accurately determined using only a few of the lower frequency modes and that it is very sensitive to damage [15,16]. Lower frequency modes are less
prone to measurement errors produced by noise. The modal flexibility matrix for the structure used to validate the ED method was determined using the structure’s natural frequencies and mass-normalized modes. The following sections describe the theory behind the modal flexibility matrix, the ED method and the damage index method.

### 4.2.1 Modal flexibility matrix

The modal flexibility matrix can be calculated using [5]:

\[
F = [\Phi] \begin{bmatrix}
\ddots & \frac{1}{\omega_j^2} & \ddots \\
\ddots & \ddots & \ddots \\
\ddots & \ddots & \ddots \\
\frac{1}{\lambda_j} & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix} [\Phi]^T = [\Phi] \begin{bmatrix}
\ddots & \frac{1}{\lambda_j} & \ddots \\
\ddots & \ddots & \ddots \\
\ddots & \ddots & \ddots \\
\frac{1}{\lambda_j} & \ddots & \ddots & \ddots & \ddots \\
\end{bmatrix} [\Phi]^T
\]

(4.1)

where \([\Phi] = [\phi_1, \phi_2, \ldots, \phi_n]\) is the mode shape matrix, \(\phi_j\) is the \(j\)-th mode shape, \(\omega_j\) are the natural frequencies per mode \(j\) in radians per second, and \(\lambda_j\) are the eigenvalues per mode \(j\). After determining the mode shapes and natural frequencies of the structure from a power spectrum analysis, the mode shapes are mass-normalized. The differential flexibility matrix, \(\Delta F\) can be determined by subtracting the undamaged modal flexibility matrix, \(F_u\) from the damaged modal flexibility matrix, \(F_d\).

### 4.2.2 Eigenparameter decomposition of structural flexibility change

#### 4.2.2.1 Approximated differential flexibility matrix.

An approximated way of determining \(\Delta F\) using the mode shapes and eigenvalues of the undamaged and damaged structure is proposed:

\[
\Delta F \approx \sum_{i=1}^{NM} \frac{1}{\lambda_{dj}} \phi_{dj} \phi_{dj}^T - \sum_{i=1}^{NM} \frac{1}{\lambda_{uj}} \phi_{uj} \phi_{uj}^T
\]

(4.2)

where \(NM\) is the number of modes, \(\phi_{dj}\) and \(\phi_{uj}\) are the damaged and undamaged eigenvectors per mode \(j\), respectively, and \(\lambda_{dj}\) and \(\lambda_{uj}\) are the damaged and undamaged eigenvalues per mode \(j\), respectively. Note that this equation describes the same procedure as subtracting the baseline modal flexibility matrix from the damage modal flexibility matrix.
4.2.2.2 Damage detection. The eigenvalue problem for the differential flexibility matrix is then solved. The number of damaged elements in the system, \( q \), is given by the number of non-zero values in the diagonal of the eigenvalue matrix of the differential flexibility matrix, \( \Lambda \).

4.2.2.3 Damage location. The eigenvalue problem can also be solved for each elemental stiffness matrix of the discretized structure. The stiffness connectivity matrix is calculated such that:

\[
C = \begin{bmatrix} c_1 & c_2 & \ldots & c_m & \ldots & c_{NE} \end{bmatrix}
\]

(4.3)

\[
c_m = \sqrt{\sigma_m u_m}
\]

(4.4)

where \( NE \) is the total number of elements, \( c_m \) is the \( m \)-th elemental stiffness connectivity vector, and \( \sigma_m \) and \( u_m \) are the non-zero eigenvalue and eigenvector of the \( m \)-th elemental stiffness matrix, respectively.

Finally, the location matrix is generated by:

\[
L = U^T F_d C
\]

(4.5)

where \( U \) is the eigenvector matrix of \( \Delta F \). The location of the damaged elements is given by the columns of \( L \) with zero value entries.

4.2.2.4 Damage quantification. If the number of damaged elements is \( q \), the columns in \( L \) corresponding to the damaged elements can be assembled into a matrix \( S \) as:

\[
S = \begin{bmatrix} l_1 & l_2 & \ldots & l_q \end{bmatrix}
\]

(4.6)

It can be shown that there exists a relationship between the eigenvalue matrix of the differential flexibility matrix and \( S \) defined by:

\[
\Lambda = S \Delta P S^T
\]

(4.7)

The matrix \( \Delta P \) is a diagonal matrix whose entries are the elemental stiffness parameters, \( \alpha_m \). Each elemental stiffness parameter represents the fraction of elemental stiffness loss due to damage, so that \( 0 \leq \alpha_m \leq 1.0 \). The greater the value of the stiffness parameter, the more severe the damage.
The relationship in Equation (4.7) still remains valid when the rows containing zero entries in $S$, and the rows and columns containing zero diagonal entries in $A$ are removed to form matrices $S^*$ and $A^*$, respectively. The damage severity can then be determined by simply solving for $\Delta P$:

$$
\Delta P = (S^*)^{-1}A^*(S^{*T})^{-1}
$$

(4.8)

In summary, the ED method determines the number of damaged elements from the number of non-zero entries in the diagonal of the eigenvalue matrix of the differential flexibility matrix, $A$. The location of the damaged elements is determined by the columns with zero value entries in the location matrix, $L$. The extent of the damage is determined by the non-zero elemental stiffness parameters in matrix $\Delta P$.

4.2.3 Automation of decision-making components in the ED method

There are three instances during the execution of this method in which the correct identification of zero value entries is critical for proper damage quantification. Zeros must be identified: (1) in the location matrix ($L$) in order to locate damage and form matrix $S$ according to Equation (4.6), (2) in the $S$ matrix in order to form matrix $S^*$, which should be a square matrix, and (3) in the eigenvalue matrix $A$ to form the square matrix $A^*$. The formation of $S^*$ and $A^*$ are essential to calculate the estimated elemental stiffness parameters according to Equation (4.8).

When attempting to run the algorithm automatically, the software can only identify a value as zero if it is exactly zero. In the great majority of cases, there will not be any exact zero entries in the diagonal of the eigenvalue matrix of the differential flexibility matrix or in the location matrix. This requires that a user initially interprets the number and location of damaged elements by inspecting the entries in the location and eigenvalue matrices. Once the damaged and undamaged elements are identified, quantification can be determined automatically. A set of formulas that turn the zero value entries into exact zeros in $L$, $S$ and $A$ is developed for programming purposes.
The columns of the location matrix that form matrix $S$ are those that contain zeros. These columns correspond to the damaged elements. Therefore, the location matrix must be modified such that:

$$ L = \frac{\text{round}(Lx)}{x} \quad (4.9) $$

where $x_1 < x < x_2$ and

$$ x_1 = \frac{0.5}{\text{min}(\text{min} |L(:, \text{undamaged elements})|)} \quad (4.10) $$

$$ x_2 = \frac{0.5}{\text{max}(\text{min} |L(:, \text{damaged elements})|)} \quad (4.11) $$

Once matrix $S$ is formed, the zero rows are removed to form $S^*$. In order to identify the zero rows, the matrix $S$ must be modified such that:

$$ S = \frac{\text{round}(Sy)}{y} \quad (4.12) $$

where $y_1 < y < y_2$ and

$$ y_1 = \frac{0.5}{\text{min}(\text{min} |S(:, \text{damaged elements})|)} \quad (4.13) $$

$$ y_2 = \frac{0.5}{\text{max}(\text{max} |S(:, \text{undamaged elements})|)} \quad (4.14) $$

Finally, the rows and columns containing zero diagonal entries in $\Lambda$ must be removed to form $\Lambda^*$. The eigenvalue matrix $\Lambda$ must be modified as:

$$ \Lambda = \frac{\text{round}(\Lambda z)}{z} \quad (4.15) $$

where $z_1 < z < z_2$ and

$$ z_1 = \frac{0.5}{\text{min} \text{diag}(\Lambda(\text{damaged elements}))} \quad (4.16) $$

$$ z_2 = \frac{0.5}{\text{max} \text{diag}(\Lambda(\text{undamaged elements}))} \quad (4.17) $$
It must be noted that in a 2-degree-of-freedom (DOF) system the lower bounds and upper bounds for \( y \) and \( z \) are interchanged, i.e. the lower bound for \( z \) is \( z_2 \) while the upper bound is \( z_1 \), and the lower bound for \( y \) is \( y_2 \) while the upper bound is \( y_1 \).

### 4.2.4 Damage index method

In the damage index (DI) method, damaged elements can be identified based on the relatively larger values of the damage indices [194,195]. A damage index, \( \beta_j \) relates the deformation of the \( j \)-th element in the \( i \)-th mode (\( \Delta_{ij} \)) with the deformation of the corresponding element and mode in the damaged structure (\( \Delta_{ij}^* \)). For \( NE \) number of elements, \( f_{ij} \) is defined by:

\[
f_{ij} = \frac{(\Delta_{ij})^2}{\sum_{j=1}^{NE}(\Delta_{ij})^2}
\]

and \( f_{ij}^* \) is the complex conjugate of \( f_{ij} \) such that:

\[
f_{ij}^* = \frac{(\Delta_{ij}^*)^2}{\sum_{j=1}^{NE}(\Delta_{ij}^*)^2}
\]

The damage index is determined by:

\[
\beta_j = \frac{f_{ij}^* + 1}{f_{ij} + 1} + 1
\]

The damage index calculated by Equation (4.20) can only be represented for one mode at a time. To take multiple modes into account, the following relationship must be used:

\[
\beta_j = \frac{(\sum_{i=1}^{NM} f_{ij}^*) + 1}{(\sum_{i=1}^{NM} f_{ij}) + 1} + 1
\]

The normalized damage index, \( Z_j \) for each element \( j \) is determined assuming the standard form of the damage index:

\[
Z_j = \frac{\beta_j - \mu_\beta}{\sigma_\beta}
\]
where $\mu_\beta$ and $\sigma_\beta$ are the mean and standard deviation of the damage indices per mode $i$. To classify an element as damaged or undamaged, a threshold $\lambda$ is established so that if $Z_j \geq \lambda$ the element is damaged and if $Z_j < \lambda$ the element is undamaged.

4.3 Methodology

4.3.1 Simulated model

A cable structure was modeled as a 2DOF system simulating a real cable structure with two acceleration sensors uniformly distributed along the cable length, as shown in Figure 4.1. Damage was simulated as a loss in bending stiffness on the second element only (shown in red) to emulate damage near the anchorage. Stiffness loss was varied in increments of 10%. The structure was modeled in Simulink/MATLAB using state-space representation. Band-limited white noise (BLWN) was input into the second DOF and the acceleration responses of both DOFs were used to perform a power spectral analysis. The signal duration was 1,000 seconds, the sampling frequency was 128 Hz and the number of points used for the Fast Fourier Transform was 16,384. The mode shapes and frequencies of the damaged and undamaged structure were determined from the frequency response functions of each DOF. These in turn were used to determine the modal flexibility matrices, $F_u$ and $F_d$. The mode shapes for both modes and the frequency response function for the second mode are shown in Figure 4.2 and Figure 4.3, respectively.

![Figure 4.1. Two-degree-of-freedom cable model analyzed.](image-url)
The ED and DI methods for damage identification were then implemented and their performance evaluated. The effect of using the exact differential flexibility matrix versus the approximate in the ED method is also examined. The effect of damping in each method was investigated by comparing results of the cable with critical damping ratios of 3%, 2%, and 1%, and no damping.

It is desired to apply damage identification strategies with as little user interference as possible. Hence, a 3DOF model of the cable structure was also analyzed in order develop general equations that contribute to the automation of the quantification part of the ED method algorithm for multiple DOF systems, as explained in Section 4.2.3.
4.3.2 Experimental model

To validate the ED method experimentally, a 5-story modifiable steel frame structure was built in the laboratory. Acceleration measurements were taken using the program SO Analyzer [196] with PCB 353B33 accelerometers [197] placed on each of the 5 floors in the direction of excitation. The structure was excited using a PCB 086C03 impact hammer [198], impacting the structure on the first and fifth floors, with two tests per excitation location. The data was collected at a sampling frequency of 1024 Hz for increased resolution using a uniform window. Tests were performed on the undamaged frame structure and two damage conditions. The damage conditions are described in Table 4.1 and shown in Figure 4.4.

Table 4.1. Damage Conditions.

<table>
<thead>
<tr>
<th>Damage scenario</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged</td>
<td>No damage</td>
</tr>
<tr>
<td>Damage 1</td>
<td>23.28% stiffness reduction in first floor</td>
</tr>
<tr>
<td>Damage 2</td>
<td>40.04% stiffness reduction in first floor</td>
</tr>
</tbody>
</table>

Figure 4.4. Damage scenarios: (a) undamaged case, (b) damage case #1, (c) damage case #2.
The natural frequencies are shown in Table 4.2 and the mode shapes in Figure 4.5. The modal parameters were mass-normalized and used to determine the modal flexibility matrix for each condition. The DI method uses the mass-normalized modes to calculate the damage indices, as described in Section 4.2.4. The threshold value $\lambda$ was taken as 1.0.

The ED method uses the elemental stiffness matrices and the modal flexibility matrices to calculate the elemental stiffness parameters as described in Section 4.2.2. The elemental stiffness matrices in global coordinates were hand calculated using the material and geometric properties of the columns in the frame structure. Since the input for this method is not analytical it is to be expected that the eigenvalue matrix of the differential flexibility matrix $A$, the location matrix $L$, and, by consequence, matrix $S$ will not contain exact zero value entries. Since the method is based on choosing non-zero entries in the diagonal of $A$ to form $A^*$, choosing columns with zero entries in $L$ in order to form $S$, and eliminating rows containing zero entries in $S$ to form $S^*$, a method to improve automation of selection of zero entries was developed, as described in Section 4.2.3.

**Table 4.2. Natural frequencies of damage case #1 and the undamaged structure.**

<table>
<thead>
<tr>
<th>Mode</th>
<th>Undamaged</th>
<th>Damaged</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18.5</td>
<td>17.5</td>
</tr>
<tr>
<td>2</td>
<td>61.5</td>
<td>58.5</td>
</tr>
<tr>
<td>3</td>
<td>103.0</td>
<td>100.0</td>
</tr>
<tr>
<td>4</td>
<td>143.0</td>
<td>140.0</td>
</tr>
<tr>
<td>5</td>
<td>172.0</td>
<td>171.0</td>
</tr>
</tbody>
</table>
4.4 Results

4.4.1 Simulation model

4.4.1.1 Eigenparameter decomposition of modal flexibility matrix method. This section illustrates the many variables that have a role in the performance of the ED method. The 2DOF system was evaluated for stiffness losses varying from 0-50% in increments of 10%. Damage detection and location were correctly established in all cases, independently of damage severity, damping ratio or the method of determining the differential flexibility matrix.

The stiffness loss estimated by the ED method using the exact differential flexibility matrix (as explained in Section 4.2.1) compared to the performance using the approximate differential flexibility matrix (Equation (4.2)) with 3% damping is shown in Figure 4.6. It can be observed that the damage extent was consistently overestimated and is more conservative as the damage is more severe. When the approximate differential flexibility matrix is used to solve the initial eigenvalue problem, the overestimation is slightly larger than when the exact differential flexibility matrix is used.

However, it was found that the relation between the exact and estimated stiffness loss is very predictable, so a mathematical relation between them can be established. This relation has been found by means of an exponential regression with a relative predictive power of $R^2 = 0.9781$ as:

![Figure 4.5. Mode shapes of damage case #1 and the undamaged structure.](image-url)
estimated loss = 7.57814e^{0.0538} exact loss \hspace{1cm} (4.23)

Therefore, the initial estimated stiffness loss can be converted to a final estimation that is very close to the exact loss, as shown in Figure 4.7. Equation (4.23) provides a reliable means of reducing the degree of overestimation intrinsic in this method.

![Figure 4.6. Estimated vs. exact cable bending stiffness loss using the approximate or exact differential flexibility matrix.](image)

![Figure 4.7. Initial and final estimations of the ED method.](image)

The final damage severity estimations for the undamped system and the system with critical damping ratios of 3%, 2%, and 1% are shown in Table 4.3. The estimation in the undamped structure is
slightly smaller than that in any of the three damped cable cases. However, it can be seen by comparing the three cases where damping is included that the amount of damping has no impact on the estimation in stiffness loss.

<table>
<thead>
<tr>
<th>Loss (%)</th>
<th>Final loss estimation by ED method (%)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ξ = 0.03</td>
<td>ξ = 0.02</td>
<td>ξ = 0.01</td>
</tr>
<tr>
<td>0</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>8.93</td>
<td>8.90</td>
</tr>
<tr>
<td>40</td>
<td>43.90</td>
<td>43.90</td>
</tr>
<tr>
<td>50</td>
<td>51.93</td>
<td>51.95</td>
</tr>
</tbody>
</table>

4.4.1.2 Damage index method. The damage indices for the case with 10% stiffness loss and 3% critical damping are shown in Figure 4.8. The values of the normalized damage indices considering the first mode, second mode or all modes are all ±0.7071 for each element. This is also true for all damage severity cases with 2% and 1% critical damping, and no damping. The normalized damage indices in the DI method have shown to be ineffective in locating damage in the cable structure with 2 DOFs. It is expected, however, that as more DOFs are considered, as in more complex structures, normalized damage indices are more trustworthy in revealing damage location since the distribution of a random variable reveals the expected value when the population is larger. This application would be impractical for a cable structure since it would require a large number of acceleration sensors to be placed.
The unnormalized indices can indicate damage more appropriately if the first mode or all modes of vibration are considered simultaneously in the analysis. When only the first mode is considered, the
damage indices for the 3% damping case with 10% stiffness loss are 1.0149 for the first element and 0.9801 for the second element. When all modes are considered, these are 1.0125 and 0.9877, respectively. These results are the most consistent with damage location, although the percentage difference between elements when all modes are considered is merely 2.52%. When only the first mode is considered, the percentage difference between elements is 3.55%. When only the second mode is considered in the analysis, the damage indices are 0.9994 for the first element and 1.0004 for the second element, which is not consistent with the location of damage. The difference in damage indices between elements is also very slight, making it impossible to determine an appropriate damage threshold, $\lambda$.

4.4.2 Experimental model

4.4.2.1 Eigenparameter decomposition of modal flexibility matrix method. The greatest challenge in using this method with real data is accounting for noise and measurement errors, which result in having none of the entries in the location matrix, $L$ or in the eigenvalue matrix, $A$ being exactly zero. In order to accomplish this in a systematic manner, the formula set described in Section 4.2.3 was developed to modify these matrices and to modify matrix $S$ in order to correctly form matrix, $S^*$ and solve for the stiffness parameter matrix, $P$ directly. When the automated ED method was used, the number and location of all damaged elements was correctly identified. Also, when using the automated ED method, the stiffness parameter of the damaged element is always within an 18.2% error for cases where the fifth floor is excited. The stiffness parameter when the first floor is excited is always within a 54.3% error. This larger error for excitation in the first floor is to be expected since the lower modes are best excited when the input force is applied at degrees of freedom where those mode shapes are displayed. The stiffness parameters for the experimental data are shown in Table 4.4 and in Table 4.5.
Table 4.4. Stiffness parameters using ED method for the experimental damage case #1.

<table>
<thead>
<tr>
<th>Damage Case Trial</th>
<th>#1 / 5th floor excitation</th>
<th>#2 / 5th floor excitation</th>
<th>#1 / 1st floor excitation</th>
<th>#2 / 1st floor excitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1 / 5th floor excitation</td>
<td>$\alpha_1 = 0.190$ (18.2% error)</td>
<td>$\alpha_5 &gt; 1^{*}$</td>
<td>$\alpha_1 = 0.191$ (18.0% error)$^{**}$</td>
<td>$\alpha_1 = 0.191$ (18.0% error)$^{**}$</td>
</tr>
<tr>
<td>#2 / 5th floor excitation</td>
<td>$\alpha_1 = 0.192$ (17.5% error)</td>
<td>$\alpha_1 = 0.192$ (17.4% error)</td>
<td>$\alpha_2 = 0.137^{*}$</td>
<td>$\alpha_3 = 0.147$</td>
</tr>
<tr>
<td>#1/ 1st floor excitation</td>
<td>$\alpha_1 = 0.111$ (52.3% error)</td>
<td>$\alpha_1 = 0.111$ (52.3% error)</td>
<td>$\alpha_1 = 0.106$ (54.3% error)$^{**}$</td>
<td>$\alpha_1 = 0.107$ (53.9% error)$^{**}$</td>
</tr>
<tr>
<td>#2 / 1st floor excitation</td>
<td>$\alpha_1 = 0.112$ (51.9% error)</td>
<td>$\alpha_1 = 0.112$ (51.9% error)</td>
<td>$\alpha_1 = 0.106$ (54.3% error)$^{**}$</td>
<td>$\alpha_1 = 0.107$ (53.9% error)$^{**}$</td>
</tr>
</tbody>
</table>

* uses automated ED method

** uses altered ED method
Table 4.5. Stiffness parameters using ED method for the experimental damage case #2.

<table>
<thead>
<tr>
<th>Damage Case Trial</th>
<th>#1 / 5th floor excitation</th>
<th>#2 / 5th floor excitation</th>
<th>#1 / 1st floor excitation</th>
<th>#2 / 1st floor excitation</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1 / 5th floor excitation</td>
<td>$\alpha_1 = 0.417$ (4.06% error)</td>
<td>$\alpha_1 = 0.417$ (4.22% error)</td>
<td><strong>uses altered ED method</strong></td>
<td><strong>uses altered ED method</strong></td>
</tr>
<tr>
<td>#2 / 5th floor excitation</td>
<td>$\alpha_3 = 0.562^*$</td>
<td>$\alpha_3 = 0.570^*$</td>
<td>$\alpha_3 = 0.574^*$</td>
<td>$\alpha_3 = 0.537^*$</td>
</tr>
<tr>
<td></td>
<td>$\alpha_3 = 0.417$ (4.12% error)**</td>
<td>$\alpha_1 = 0.418$ (4.27% error)**</td>
<td>$\alpha_1 = 0.460$ (14.9% error)**</td>
<td>$\alpha_1 = 0.460$ (15.0% error)**</td>
</tr>
</tbody>
</table>

* uses automated ED method

** uses altered ED method

4.4.2.2 Damage index method. The DI method was successful in determining the damage location for both of the damage cases independently of the location of excitation. The damage indices are shown in Figure 4.9, where the normalized damage index for the first floor is always greater than $\lambda = 1.0$ and all normalized damage indices for the other floors are less than $\lambda = 1.0.$
4.5 Conclusions

The automated ED method has shown to be effective in rapidly determining the existence, location and severity of damage near simulated cable anchor zones, with no false indications. Damage location is always correctly determined with the automated ED method, while the DI method does not always clearly reveal damage location. Although the initial damage severity estimation with the ED method is very conservative, especially when damage is more severe, an exponential expression can be used to determine a final estimation that resembles the real damage severity quite effectively. This method was effective to quantify the damage in elements of a laboratory-scale experiment, especially when the input was placed at the fifth floor of a 5-story shear building. The steps proposed for the automation of the algorithm make damage quantification much faster and accurate than the unaltered method. These findings and
developments represent a step towards the implementation of the ED method for damage identification in real cable structures.
Chapter 5 Rapid Vibration-based Cable Tension Estimation Using Wireless Smart Sensing

5.1 Introduction

Contrary to most civil structures, bridges experience highly variable loading conditions that result in unique member behavior throughout their lifespan. Loading condition can be affected by cyclic vehicle loading and natural hazards. Structural loading capacity can also be changed significantly by harsh environmental conditions. This is particularly sensitive when the loading capacity of critical load-carrying members is reduced. Cable-supported bridges rely on the condition of stay cables to transmit loads effectively without compromising the safety of users. It follows then that the determination of tension force of cables in cable-supported bridges is of special interest.

Cable loading capacity can fluctuate significantly due to changes in external loading, cable relaxation or damage existence, whether on the member in question or in other members. Stay cables are especially susceptible to damage. Cables are exposed cyclic changes in load pattern and wind-induced buffeting that cause fatigue damage [102,103]. The exposure of cables to high levels of moisture can also result in severe corrosion, reducing cable diameters up to 30% [5]. This translates into a loss in bending stiffness of up to 76%, and hence a significant reduction in loading capacity. When loading capacity is significantly reduced, other members may undergo excessive loading to compensate in order to complete the load path effectively. These changes in loading condition could potentially exceed member loading capacity, raising a safety concern. This stresses the importance of determining cable tension force effectively.

To this end, many researchers have studied various methods to determine cable tension. One of the most common ways to directly determine cable tension is using lift-off tests [104]. In a lift-off test, a load cell and a hydraulic jack placed in series are wedged into a required space in an anchor block where the cables are secured. This method requires a space for the inclusion of a hydraulic jack in the original design of the bridge and is very costly to perform. On the other hand, other sensors such as strain gauges
offer a less invasive and expensive approach to measure changes in displacement [93]. However, strain gauges can be susceptible to environmental conditions. They are also challenging to install and require direct contact with cable strands, which are not always exposed due to sheathing protection. Other strain measuring methods use Fiber Bragg grating sensors, which offer a high accuracy strain sensing option based on optical fiber that can overcome some of the limitations of strain gauges, such as the susceptibility to environmental conditions [105]. Yet, the installation of this technology remains complex and is significantly more expensive than strain gauges. Electromagnetic sensors can also be used to accurately determine cable tension, but they are even more complex to install and require a sensing coil to be turned around the unsheathed cable strand [104]. Direct measurement methods are thus labor-intensive and costly, especially when compared to vibration-based methods.

Vibration-based measurements provide modal information that can be used for multiple monitoring objectives at a much lower cost. Most vibration-based tension estimating methods use taut string theory, developed by Max Irvine in 1981 [106]. This method uses wave propagation theory to determine a simple formula using geometric and material properties to determine tension in a cable based on its natural frequencies. Several modifications to the basic formula have been added since to include bending stiffness effects, sag, varying cable mass, and end conditions [107,108]. In addition, empirical corrections and finite element-based model updating has enhanced these formulations to improve tension accuracy [109–112]. Some of these tension estimation methods have been used as algorithms in multiple developments for vibration-based cable tension estimation. Kangas et al. [63] used finite-difference model developed by Mehrabi and Tabatabai [108] to demonstrate that ambient vibration is sufficient to determine an accurate estimation of cable tension force in a cable-stayed bridge, even when the capacitive accelerometers used collect data from the cable sheath. Cho et al. [64] used tension estimating formulas developed by Zui et al. [107] considering cable sag and flexural rigidity as embedded modules in an in-house wireless smart sensor development. Results were comparable to tension measurements determined with strain gauges. Jung et al. [49] used extended taut string formulas considering damping ratio, sag, and bending stiffness as presented by Kim and Park [111] to determine cable tension in an inclined full-scale
cable in a laboratory environment with great accuracy. These applications have demonstrated the practicality of vibration-based tension estimations methods. However, further development in affordable, wireless, and expandable structural health monitoring (SHM) systems that can provide reliable tension estimation are required.

A low-cost SHM station that can perform rapid tension force estimation is detailed in this chapter. Using a remotely controlled commercial single-board microcomputer with a universal serial bus (USB) accelerometer, the acceleration time history of cables can be recorded and all spectral analysis and tension estimation can be performed within the microcomputer. A power spectral density (PSD) function and four vibration-based tension force estimation algorithms were programmed in embedded Wolfram packages that can be executed remotely. The software also allows ease of onboard modification and expansion in the algorithms making post-processing possible if needed. The microcomputer board contains additional sensors and ports that allow expanded monitoring capability, such as humidity, temperature, video, and external device control. These sensors can be used to collect environmental information and even perform vehicle counting [199]. This wireless, smart sensing system can also be installed with great ease. The entire system was validated on a laboratory-scale elevator cable subjected to a constant load, and the onboard packages were additionally validated with acceleration data from a through arch bridge in Middletown, Connecticut. The smart wireless system communicated remotely in an efficient manner during the laboratory test. It was successful at initiating, collecting, saving and processing the acceleration signal of the vibrating elevator cable. Excellent agreement was found between the applied tension in the cable and the tensions estimated using the vibration-based methods. The estimations determined in a previous study on the through arch bridge data were comparable to those determined by the onboard tension estimating algorithms, and corroborated the assessment of the methods during the laboratory-scale test. This wireless smart system has great potential for reliable cable tension estimation in the field, while providing simple expandability options for additional SHM monitoring applications at a low cost.
5.2 Cable tension estimating vibration-based methods

This section summarizes the theory and assumptions of the vibration-based tension estimation algorithms programmed on the smart sensing system. The theory of the methods explained in this section is valid for vertical bridge hangers with hinged connections, which was the type of structure on which the wireless cable tension estimating system was tested. All methods originate from taut string theory [106], which is presented as the first estimation method. The second method consists of the widely used practical formulas presented by Zui et al. [107]. The third method is a set of empirical formulas presented by Ren et al. [109] where least square fitting is performed to adjust theoretical fundamental frequency and theoretical tension of the cable considering bending stiffness and sag effects separately. The fourth method is a different set of practical formulas presented by Huang et al. [110] that were determined by applying correction coefficients to general solutions of governing vibration equations based on numerical analysis results. This method expands modern cable theory by considering bending stiffness but neglecting sag extensibility [106,111,200]. The method was chosen because bending stiffness would play a more significant role than sag extensibility in the application of bridge hangers.

5.2.1 Taut string theory

When taut string theory [106] is applied to a vertical suspended cable subjected to free vibrations, the cable can be modeled as a vertical string of linear mass, \( m \) and length, \( L \) pinned at both ends, as shown in Figure 5.1.

![Figure 5.1 Taut string model of a suspension cable.](image)
Assuming the string is tensed with a tension force $T$, the wave propagation equation for equilibrium is:

$$m \frac{\partial^2 x}{\partial t^2} - T \frac{\partial^2 x}{\partial y^2} = 0$$

(5.1)

Equation (5.1) can be rewritten in terms of the wave speed, $v$ and using a variable $u = y - vt$. The resulting wave equation becomes:

$$\frac{\partial^2 f}{\partial u^2} (mv^2 - T) = 0$$

(5.2)

where $f$ is the generalized traveling wave representation of $x(y,t)$. In order to attain a non-trivial solution, the term in parenthesis in Equation (5.2) must be equal to zero. Assuming that the shape of vibration is harmonic, the wave speed can also be expressed as $v = A_n f_n$, where $f_n$ is the natural frequency of the $n$-th mode of vibration and $A_n$ is the wavelength at the $n$-th mode of vibration equal to $2L/n$. Substituting the wavelength and the natural frequency into Equation (5.2) and solving for tension:

$$T = 4mL^2 \left( \frac{f_n}{n} \right)^2$$

(5.3)

Since cable length and linear mass are presumably known quantities, tension can be determined using modal information extracted from acceleration measurements via spectral analysis. Sometimes, multiple modes can be extracted and some modes are represented more clearly than others. Equation (5.3) implies that the ratio $f_n/n$ is constant, since the tension in the cable is assumed to be a constant value. If several natural frequencies determined experimentally are linearly fitted, the slope of the equation will represent the ratio $f_n/n$ for $n = 1$, which is equal to the fitted fundamental frequency of the cable structure. Hence, the final estimation of the tension force for the taut string model of the cable follows a harmonic assumption and it follows that:

$$T = 4mL^2 f_{1, fit}^2$$

(5.4)
5.2.2 Practical formulas for cable tension estimation

5.2.2.1 Method 1. The most widely used and validated set of formulas for vibration-based cable tension estimation are those presented by Zui et al. [107]. Most vibration-based cable tension estimation methods that have been developed after these practical formulas compare their validation to that of this method, making it a benchmark for future algorithms. These formulas are based on an extended taut string model to include additional geometric characteristics, such as bending stiffness and sag extensibility, and are adjusted to empirical data.

It was determined that the usage of the fundamental frequency, \( f_i \) for tension estimation of a cable with sufficiently small sag yields more accurate results than when higher-order modes are used. In the programmed algorithm, the fitted fundamental frequency, \( f_{i,\text{fit}} \) is used for calculation. A distinct tension formula is chosen based on a dimensionless parameter that quantifies the relevance of bending stiffness in cable behavior. This parameter is defined as:

\[
\xi = \sqrt{\frac{T}{EI}} \frac{1}{L} \quad (5.5)
\]

where \( E \) is the elastic modulus and \( I \) is the cross-sectional moment of inertia. The formula set for cables with small sag is the following:

\[
T = 4m (f_{1,\text{fit}} L)^2 \left[ 1 - 2.20 \frac{C}{f_{1,\text{fit}}} - 0.550 \left( \frac{C}{f_{1,\text{fit}}} \right)^2 \right] ; \quad \xi \geq 17 \quad (5.6)
\]

\[
T = 4m (f_{1,\text{fit}} L)^2 \left[ 0.865 - 11.6 \left( \frac{C}{f_{1,\text{fit}}} \right)^2 \right] ; \quad 6 \leq \xi \leq 17 \quad (5.7)
\]

\[
T = 4m (f_{1,\text{fit}} L)^2 \left[ 0.828 - 10.5 \left( \frac{C}{f_{1,\text{fit}}} \right)^2 \right] ; \quad 0 \leq \xi \leq 6 \quad (5.8)
\]

where the parameter \( C \) is defined as:

\[
C = \sqrt{\frac{EI}{mL^4}} \quad (5.9)
\]
The dependence of $\xi$ on the applied tension force on the cable (see Equation (5.5)) indicates that this method is recursive and requires an initial guess of tension force. The method converges when the newest calculation of $\xi$ falls in the same range as the previous calculation according to Equations (5.6) - (5.8).

5.2.2.2 Method 2. The set of empirical formulas presented by Ren et al. [109] considers bending stiffness and sag influence separately. These formulas include coefficients derived with least square fitting to theoretical data and they all depend on the fundamental frequency of the cable. As explained for the previous methods, the fitted fundamental frequency is used instead of the measured fundamental frequency in the programmed algorithms. As in the formula set of Method 1, formula selection in this method depends on the range in which the non-dimensional parameter for bending stiffness, $\xi$, falls for cables where sag is negligible. The formulas in this method are the following:

$$T = 3.432mL^2f_{1,\text{fit}}^2 - 45.191 \frac{EI}{L^2}; \quad 0 \leq \xi \leq 18 \quad (5.10)$$

$$T = m \left(2Lf_{1,\text{fit}} - \frac{2.363}{L} \sqrt{\frac{EI}{m}} \right); \quad 18 < \xi \leq 210 \quad (5.11)$$

$$T = 4mL^2f_{1,\text{fit}}^2; \quad \xi > 210 \quad (5.12)$$

5.2.2.3 Method 3. The third set of practical formulas programmed in the wireless smart system was developed by Huang et al. [110]. This method is based on form-finding theory and a new catenary element for form finding of suspension cables. These formulas can be applied to cables with different boundary conditions (simply supported ends, fixed-fixed ends, and hinged-fixed ends) using frequencies of the first 10 modes of vibration. They are applicable to cables with large sag-extensibility when using second- or higher-order modes. In general, according to the authors’ results, the usage of higher-order mode results in better tension estimation of cables with relatively large $\xi$. This set of practical formulas was determined by applying correction coefficients ($K_n$) to general solutions of governing vibration equations based on numerical results. The unified formula applicable to all cases is:
\[ T = K_n T_s \]  

(5.13)

where \( T_s \) is the tension estimated according to taut string theory equal to Equation (5.3) and \( K_n \) is a numerically determined parameter according to the boundary condition case. Since this method uses numerically adjusted coefficients to modify tension defined according to taut string theory, the natural frequencies used to determine \( T_s \) will be linearly fitted. The general solution for \( K_n \) is:

\[ K_n = -A_n \lambda_n^2 - B_n \lambda_n + 1 \]  

(5.14)

where \( \lambda_n \) is the non-dimensional form of the bending stiffness:

\[ \lambda_n = \frac{EI}{\sqrt{4\pi^2 mL^4 f_{n,fit}^2}} \]  

(5.15)

Substituting Equations (5.3), (5.14), and (5.15) into Equation (5.13), a general and expanded tension estimation equation is obtained:

\[ T = -A_n \frac{EI}{(Ln\pi)^2} - B_n \frac{2f_{n,fit}\sqrt{Elm}}{\pi n^2} + 4mL^2 \left( \frac{f_{n,fit}}{n} \right)^2 \]  

(5.16)

The coefficients \( A_n \) and \( B_n \) are determined according to the boundary conditions of the cable. This allows tension estimation when the boundary conditions of the cable are anything other than hinged, such as when shock absorbers are used in cable groups. However, both the laboratory-scale cable and the cables on the suspension bridge used to validate the wireless tension estimation system have hinged-hinged supported ends, so only this case is shown below. The coefficients \( A_n \) and \( B_n \) in this boundary condition case are:

\[ \begin{align*}
A_n &= (n\pi)^4 \\
B_n &= 0
\end{align*} \]  

(5.17)

Substituting Equation (5.17) into Equation (5.16), the tension estimation formula for the hinged-hinged boundary condition becomes:

\[ T = 4mL^2 \left( \frac{f_{n,fit}}{n} \right)^2 - \frac{EI}{L^2} (n\pi)^2 \]  

(5.18)
Tension in this method was determined using higher-order mode frequencies in addition to the fundamental frequency, since Huang et al. [110] report more accurate tension estimation results when higher-order modes are used in cables with high $\xi$.

### 5.3 Wireless smart sensing system

In order to quickly estimate tension force in suspension cables, a wireless tension estimating system has been developed. The system consists of off-the-shelf, commercially available hardware and software packages. It includes a single-board computer with a USB accelerometer that is capable of acquiring real-time data stream while connected to a USB port via the USB human interface device (HID) protocol. The microcomputer is remotely controlled via a secure shell (SSH) connection or a virtual network computing (VNC) connection to a remote computer. This connection is established through a wireless local area network (WLAN). The following sections provide details on the components of the tension estimating system.

#### 5.3.1 Hardware

The first relevant hardware component in this tension estimation system is the acceleration sensor. The USB accelerometer model X2-2 developed by Gulf Coast Data Concepts, LLC is a data logger that contains the high sensitivity, low noise, triaxial acceleration sensor Kionix KXR-2050 [201]. It can record acceleration data ranging ±1.25g or ±2g for high or low gain, respectively. The logger is developed especially for structural vibration and earthquake monitoring applications, with an operational temperature range of -5°F to 130°F (-20°C to 55°C). With a 15 bit resolution, the user can choose the sampling rate as 8, 16, 32, 64, 128, 256, or 512 Hz. The logger shown in Figure 5.2 is 4.10 in long, 1.01 in wide, and 1.04 in high.
The Raspberry Pi (RPi) 3 Model B is a fully functional, low-cost, single-board microcomputer that has been developed for affordable programming experimentation with educational purposes. The board has a 1.2GHz quad-core ARM Cortex-A8 CPU and 1GB of random-access memory (RAM). The RPi microcomputer was chosen due to its low-cost and versatility. The RPi board is equipped with a temperature and humidity sensor that allows programmability of weather tracking. It also allows the inclusion of a low-cost camera with extensive programmable features applicable to video monitoring. The RPi board also includes 40 general purpose input-output (GPIO) pins to allow further cyber-physical interactions between the microcomputer and the physical environment (see Figure 5.3). With 4 USB ports and ample RAM, the RPi board allows the realtime streaming of the USB X2-2 accelerometer. Using the built-in chip antenna, the RPi can connect to a remote host computer and be directly controlled via the computer’s interface, which reduces the need of additional on-site equipment to a laptop. The adaptability of the RPi via the variety of modes and accessories makes it useful for remote multifunction monitoring.
5.3.2 Software

Data logs written by the X2-2 USB accelerometer are compatible with several operation systems, including Linux OS. The data logger supports USB HID class specification for computer peripherals to allow real-time data stream to the microcomputer. In order to access the capability, the gcdcTool command line utility was installed in the microcomputer. This software allows to stream data, update firmware, synchronize the real-time clock and check system status.

The board runs on the Raspbian Jessie operating system, a freely distributed software based on Debian Linux. The tightvncserver tool was installed and configured in the RPi computer to allow graphical desktop sharing on the remote host computer. This allows direct access to control all functions on the RPi computer from the host computer, whether through the LXTerminal or directly on the graphical user interface. Alternatively, remote control can be performed via SSH, which provides a secure and encrypted channel of communication while operating a single application at once and maintaining more free memory as it need not generate a graphical environment. This connection can be established either using the application Terminal for Mac OS or PuTTY for Windows OS.

Onboard processing of the acceleration signal collected by the X2-2 USB accelerometer and cable tension calculation is possible due to the Wolfram application included in Raspbian Jessie, which
can be executed via a remote SSH connection. The process of analysis of the acceleration signal and onboard tension calculation is summarized in Figure 5.4. The user first defines all input values in a Wolfram package named “inputs.m”, including the local directory and file name of the acceleration signal, the overlap of the PSD windows, number of points for Fourier transform (NFFT), sampling frequency ($f_s$), and the correction factor for selected gain in the X2-2 accelerometer. The cable properties ($A, I, E, L, mg, sag, and inclination$), expected tension as the initial guess for iterations, and boundary conditions are also defined by the user in this package. Next, the package “PSD.m” is used to calculate the PSD of the acceleration signal and select the natural frequencies of the monitored cable using an automatic peak-picking function. This peak-picking function uses a user-defined parameter equal to the standard deviation of Gaussian blurring. The standard deviation of Gaussian blurring is used in image processing to blur images using a Gaussian function [202]. Using a higher standard deviation blurs the image more than using a low standard deviation. The parameter in Wolfram operates in a similar manner with peak selection, selecting peaks more close together with a low standard deviation of Gaussian blurring and selecting peaks farther apart with a high standard deviation. Once the peaks are selected, the user may view the selected natural frequencies as an output on the SSH window. To ensure the quality of the selection, a file of the PSD plot with marked selected peaks is also generated and can be obtained via secure file transfer protocol (SFTP). The user may return to “inputs.m” and change the PSD and peak-selection parameters if the peak selection or PSD function are not satisfactory. Else, the user may chose the final natural frequencies in “Freq.m” and execute the cable tension estimation packages. These final packages contain algorithms to perform linear regression analysis of the natural frequencies to determine fitted natural frequencies according to the harmonic cable behavior assumption in “linreg.m”, and to calculate tension according to taut string theory (“tautstring.m”), Method 1 (“cabletensionzui.m”), Method 2 (“cabletensionren.m”), and Method 3 (“cabletensionhuang.m”), as described in Section 5.2.
5.3.3 Communication

Communication and authentication are key to successfully operate this wireless smart system. The RPi must be capable of connecting to the local network of interest. Wi-Fi was chosen in the current study for its increased speed, stability, robustness, and communication range [203], but Bluetooth connection and others are also possible. Connection to WPA2 enterprise wireless networks require additional configuration changes to the wpa_supplicant configuration file, but ensure a more secure and private connection. Both the remote and host computer must be connected to the WLAN in order to communicate.

Authentication in order to establish SSH, SFTP and optionally VNC connections are desirable and are used in this wireless system for increased security. The SSH connection was preferred and used in this study to configure, synchronize, and execute the X2-2 accelerometer. This connection was also used to select user-defined parameters in the X2-2 data logger, including the sampling frequency and gain. The
SSH connection is also used to edit the input Wolfram package, “inputs.m” to define PSD parameters and cable properties. A second SSH connection was used to execute commands in Wolfram to run the packages defined in Figure 5.4. In order to retrieve the resulting files from data collection and onboard analysis, an SFTP connection was established between the remote computer and the RPi. In order to facilitate the transfer of multiple files simultaneously, the application FileZilla was used to establish the SFTP connection and verify onboard analysis results on the remote computer.

5.4 Experiments

Two experiments were designed and carried out to evaluate the performance of the smart wireless cable tension estimating system. The first was an impact hammer test performed in the laboratory on an elevator cable with a constant applied tension force. The second was an additional validation of the cable tension estimation methods selected using ambient vibration data from the Arrigoni Bridge, a truss arch suspension bridge in Middletown, Connecticut.

5.4.1 Laboratory elevator cable tests

Impact hammer tests on a laboratory-scale cable were conducted for validation. A cable was vertically suspended on an Instron® Satec hydraulic universal test machine with a 400 kip load cell, as shown in Figure 5.5. The cable ends were attached using a wedge-socket system. The tested cable was an 8x19 sisal fiber core NFC 1370/1770 elevator cable with a diameter of 0.5 in (12.7 mm) fabricated by Brugg Lifting [204]. The cable length between the sockets was 8.5 ft; the linear weight was 0.363 lb/ft (0.54 kg/m); the elastic modulus was 11603 ksi (80 N/mm²); the moment of inertia was 0.00156 in⁴ (651 mm⁴); and the cross-sectional area was 0.0880 in² (57.8 mm²). The cross-sectional area and moment of inertia used were based on the metallic portion of the cross-section, excluding the fiber core. A diagram of the cable cross section can be seen in Figure 5.6.
Cable tension was applied by ramping the force on the cable to 1 kip for 60 seconds and holding the applied tension for 105 seconds. The acceleration sensor was initiated 90 seconds after the tensing.
procedure was started to allow sufficient time for the applied tension force to become stable. Figure 5.7 shows the time history of the applied force. The data between the black vertical lines shows the period during which the impact was applied and the acceleration time history was collected.

![Figure 5.7. Time history of applied tension force.](image)

A close-up in Figure 5.5 shows the location of the X2-2 accelerometer, which was attached at 1/5 of the length of the cable from the edge of the bottom socket. Assuming that the shape of the vibration cable is sinusoidal, the point where the frequency content of the first four modes is collectively largest is at 1/5 of the length of the cable from one of the ends, as shown in Figure 5.8. The accelerometer was oriented such that the Z-axis (see Figure 5.2) was aligned with the direction of excitation. A PCB 086C03 impact hammer with a 084B03 hard tip was used for the input excitation.
The impact location was at 0.45 of the total cable length from the edge of the bottom socket in the Z direction of the X2-2 accelerometer (shown in Figure 5.5). The X2-2 accelerometer was configured for low gain (range of ±2g) and sampling frequency of 256 Hz. Acceleration was recorded during 48 seconds (12288 points of data). Figure 5.9 shows a typical acceleration time history. Modal impact hammer testing was performed 10 times.
The wireless tension estimating system was used to initiate, collect, save, and analyze the acceleration signal to determine the natural frequencies of the cable. The system was also used to calculate the estimated tensions. Figure 5.10 shows a screen shot of the remote computer used to control the system after all cable tension estimations were remotely calculated. All sensing and analysis can be performed through 2 SSH windows and 1 SFTP window, showing the simplicity of using the designed wireless tension estimating system. First, the system was connected to the same Wi-Fi network as the remote computer. Once an SSH connection was established, the settings of the X2-2 accelerometer were modified in the configuration file of the data logger where the sampling rate was set to 256 Hz and the gain to “low”. The Wolfram package “inputs.m” was also modified through the SSH connection to contain the parameters for the spectral analysis and the cable properties. Once the applied force on the cable had become stable, the accelerometer was initiated to collect raw data by executing the gcdcTool application. Immediately after, the cable was impacted with the hammer.

Figure 5.10. Remote computer environment while using the cable tension estimating system.
5.4.2 Arrigoni Bridge

In order to determine whether the cable tension estimation system would be effective in the field, the Arrigoni Bridge was selected to validate the system with field data. This structure is a steel truss arch suspension bridge in Middletown, Connecticut crossing the Connecticut River (see Figure 5.11). The arrangement of cables and suspender numbers within the cable groups is shown in Figure 5.12. Acceleration data was recorded on March 8, 2011 as part of a cable monitoring project [205]. In this previous publication, tension estimations were performed using taut string theory only. These previous results are compared to the tension estimations performed by the present tension estimation system, which include cable bending stiffness effects and empirical adjustments. The south side of the eastern span was selected for the experiment. Each side consists of 17 groups of 4 cables. Since each arch is symmetrical, cable groups L2 – L10 were examined. The steel cable strands have a diameter of 1.625 in. According to the ASTM A586 Standard [206], these cables have a linear weight of 5.41 lb/ft, which is a linear mass of 0.168lb-s²/ft². Table 5.1 summarizes the lengths of the cable groups.

![Map of Arrigoni Bridge](image1)

![Satellite view of Arrigoni Bridge](image2)

(a) (b)

Figure 5.11. Arrigoni Bridge in Middletown, Connecticut: (a) satellite view, (b) southwestern street view.
Figure 5.12. Elevation of the southeastern span of the Arrigoni Bridge [205].

Table 5.1. Cable lengths of the Arrigoni Bridge.

<table>
<thead>
<tr>
<th>Cable group</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
<th>L8</th>
<th>L9</th>
<th>L10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (ft)</td>
<td>11</td>
<td>26</td>
<td>39</td>
<td>50</td>
<td>59</td>
<td>66</td>
<td>71</td>
<td>74</td>
<td>75</td>
</tr>
</tbody>
</table>

The acceleration data was collected at a sampling rate of 100 Hz during 60 seconds under normal in-service ambient conditions. A Bridge Diagnostics, Inc. STI wireless system was used, consisting of one STS-Wi-Fi base station, three STS-Wi-Fi nodes, twelve 50g accelerometers and a laptop PC. The accelerometers were attached to the cables with Velcro straps at 9.5 ft from the deck and connected to the STS-Wi-Fi nodes. The nodes transmitted the data to the base station wirelessly, where the data was aggregated and transmitted to the PC for viewing and storage. The accelerometers were oriented in the north-south direction (transverse to the deck). The original study analyzed the time histories using a Hamming window with 256 data points and 90% overlap. Raw acceleration data measured from this
bridge was utilized for cable tension estimation with the new wireless cable tension estimation system instead of the windowed and averaged signal of the previous study.

5.5 Results

This section shows the results of the laboratory-scale experiment and the field data experiment after the acceleration signals were collected. The signal analyses in frequency domain and cable tension calculation results are shown next.

5.5.1 Laboratory elevator cable tests

Once the signal was collected, “inputs.m” was executed. The signal was then modified using an exponential window with a decay parameter of 7.5% and zero padding to 256 seconds. A fast Fourier transform (FFT) with a single window was calculated for the data, the result for the first trial shown in Figure 5.13. This was done by executing the “PSD.m” Wolfram package through a second SSH connection, where the Wolfram Kernel had been opened using the command “wolfram”. Figure 5.13 shows that there are two unexpected local modes present in the acceleration collection at approximately 15 and 30 Hz. The expected fundamental frequency for the applied tension of 1 kip according to taut string theory is 17.52 Hz. Since taut string theory is widely known to overestimate cable tension for measured frequencies, it is consequently true that, for a known tension, the expected frequency should be higher than that predicted by taut string theory. Therefore, the 15 Hz peak was ignored and the peak at approximately 23 Hz was assumed to be the measured fundamental frequency. The same two peaks were observed in all 10 trials, and they can also be observed from the Fast Fourier transforms of the transversal acceleration signals, such as that of the first trial shown in Figure 5.14. The consistent appearance of these peaks confirms that these salient frequencies are not due to environmental error, but are produced by some unexpected structural vibration interaction. Since the behavior of a cable structure resembles harmonic behavior, these peaks at 15 and 30 Hz are more likely caused by local modes of vibration produced by the mounting system of the cable structure or by the low frequency vibrations of the
universal testing machine, and not by the cable structure itself. However, further testing of the mounted cable system would be required to verify this, including additional impact hammer tests with more sensors installed along the length of the cable. The FFT and acceleration time history were verified by transferring the plot files generated by “PSD.m” to the remote computer via SFTP. Finally, after the measured natural frequencies were selected and input into “Freq.m”, the package “linreg.m” was executed to perform the linear regression of the natural frequencies and the cable estimation packages were run. Cable tension results were displayed through the SSH connection window.

![Figure 5.13. Fast Fourier transform of acceleration signal.](image1)

![Figure 5.14. Fast Fourier transform of transverse acceleration signal.](image2)
The linear regression of the natural frequencies for this trial is shown in Figure 5.15. The measured natural frequencies for this cable structure found in each trial and their fitted fundamental frequencies are shown in Table 5.2. All coefficients of determination ($R^2$) are very close to 1.0, showing that the linear models for the frequencies are proper fits.

![Linear regression of the natural frequencies.](image)

**Figure 5.15.** Linear regression of the natural frequencies.

**Table 5.2.** Measured natural frequencies and fitted fundamental frequencies of the cable.

<table>
<thead>
<tr>
<th>Trial</th>
<th>$f_1$ (Hz)</th>
<th>$f_2$ (Hz)</th>
<th>$f_3$ (Hz)</th>
<th>$f_4$ (Hz)</th>
<th>$f_{n,\text{fit}}$ (Hz)</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23.29</td>
<td>39.68</td>
<td>57.10</td>
<td>77.00</td>
<td>19.40</td>
<td>0.989</td>
</tr>
<tr>
<td>2</td>
<td>23.30</td>
<td>39.03</td>
<td>57.20</td>
<td>76.71</td>
<td>19.33</td>
<td>0.989</td>
</tr>
<tr>
<td>3</td>
<td>23.30</td>
<td>39.01</td>
<td>57.05</td>
<td>76.64</td>
<td>19.30</td>
<td>0.989</td>
</tr>
<tr>
<td>4</td>
<td>23.28</td>
<td>39.05</td>
<td>57.22</td>
<td>76.77</td>
<td>19.34</td>
<td>0.990</td>
</tr>
<tr>
<td>5</td>
<td>23.33</td>
<td>39.18</td>
<td>57.23</td>
<td>76.90</td>
<td>19.37</td>
<td>0.989</td>
</tr>
<tr>
<td>6</td>
<td>23.39</td>
<td>39.39</td>
<td>57.54</td>
<td>77.32</td>
<td>19.47</td>
<td>0.990</td>
</tr>
<tr>
<td>7</td>
<td>23.21</td>
<td>39.07</td>
<td>57.20</td>
<td>76.79</td>
<td>19.34</td>
<td>0.990</td>
</tr>
<tr>
<td>8</td>
<td>23.32</td>
<td>39.38</td>
<td>55.14</td>
<td>77.14</td>
<td>19.20</td>
<td>0.985</td>
</tr>
<tr>
<td>9</td>
<td>23.21</td>
<td>39.15</td>
<td>54.98</td>
<td>76.68</td>
<td>19.11</td>
<td>0.985</td>
</tr>
<tr>
<td>10</td>
<td>23.35</td>
<td>39.59</td>
<td>55.32</td>
<td>77.83</td>
<td>19.33</td>
<td>0.985</td>
</tr>
</tbody>
</table>
Tension estimations were calculated via the methods of taut string theory and the practical formulas of Methods 1-3 as described in Section 5.2. The estimated cable tension in Trial 1 using the four methods is summarized in Figure 5.16. As was to be expected, the estimations calculated using taut string theory alone were significantly conservative compared to the applied tension force. Since the end conditions in this cable were hinged-hinged, Equation (5.18) was used for the estimations of Method 3, which adds a modification to taut string theory considering bending stiffness effects. Although still conservative when using the fundamental frequency, these estimations are closer to the applied tension, particularly when considering higher-mode frequencies. The estimations provided by Methods 1 and 2 were significantly closer to the applied force, never once differing to the applied force by more than 3.65% in any trial. These two methods consider bending stiffness and the formulas are empirically adjusted to represent a real cable as opposed to the other methods, which rely on analytical and numerical developments only. The detailed force calculation and percentage differences to the applied forces are summarized in Table 5.3 and in Table 5.4. The cable tension estimation system proved reliable and effective to collect and analyze vibration data and to accurately calculate cable tension in the laboratory.

<table>
<thead>
<tr>
<th>Method</th>
<th>Percentage Difference to Applied Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method 1</td>
<td>3.65%</td>
</tr>
<tr>
<td>Method 2</td>
<td>3.65%</td>
</tr>
<tr>
<td>Method 3 - (f_1,\text{fit})</td>
<td>4.35%</td>
</tr>
<tr>
<td>Method 3 - (f_4,\text{fit})</td>
<td>5.23%</td>
</tr>
</tbody>
</table>

**Figure 5.16. Cable tension estimation in Trial 1.**
Table 5.3. Cable tension estimated in Trial 1 compared to applied tension.

<table>
<thead>
<tr>
<th>#</th>
<th>Applied force (kip)</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3 (f1, fit)</th>
<th>Method 3 (f4, fit)</th>
<th>Taut String Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.983</td>
<td>1.019</td>
<td>1.017</td>
<td>1.209</td>
<td>0.951</td>
<td>1.226</td>
</tr>
<tr>
<td>2</td>
<td>1.000</td>
<td>1.011</td>
<td>1.009</td>
<td>1.200</td>
<td>0.942</td>
<td>1.217</td>
</tr>
<tr>
<td>3</td>
<td>0.989</td>
<td>1.008</td>
<td>1.006</td>
<td>1.197</td>
<td>0.939</td>
<td>1.214</td>
</tr>
<tr>
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<td>1.012</td>
<td>1.010</td>
<td>1.201</td>
<td>0.943</td>
<td>1.218</td>
</tr>
<tr>
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<td>1.014</td>
<td>1.205</td>
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<td>1.027</td>
<td>1.026</td>
<td>1.218</td>
<td>0.960</td>
<td>1.235</td>
</tr>
<tr>
<td>7</td>
<td>0.995</td>
<td>1.012</td>
<td>1.010</td>
<td>1.201</td>
<td>0.943</td>
<td>1.218</td>
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<td>0.995</td>
<td>1.184</td>
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<td>1.009</td>
<td>1.200</td>
<td>0.942</td>
<td>1.217</td>
</tr>
</tbody>
</table>

% diff

<table>
<thead>
<tr>
<th>#</th>
<th>Applied force (kip)</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3 (f1, fit)</th>
<th>Method 3 (f4, fit)</th>
<th>Taut String Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Force (kip)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
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<td>3.65</td>
<td>3.50</td>
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<td>0.92</td>
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<td>21.70</td>
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<tr>
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<td>1.57</td>
<td>20.71</td>
<td>-5.10</td>
<td>22.43</td>
</tr>
<tr>
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<td>1.004</td>
<td>2.30</td>
<td>2.15</td>
<td>21.29</td>
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<tr>
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<td>0.995</td>
<td>1.69</td>
<td>1.54</td>
<td>20.71</td>
<td>-5.18</td>
<td>22.43</td>
</tr>
<tr>
<td>8</td>
<td>1.002</td>
<td>-0.57</td>
<td>-0.71</td>
<td>18.17</td>
<td>-7.53</td>
<td>19.89</td>
</tr>
<tr>
<td>9</td>
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<tr>
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<td>0.42</td>
<td>19.38</td>
<td>-6.25</td>
<td>21.09</td>
</tr>
</tbody>
</table>
5.5.2 Arrigoni Bridge

For the validation in the cable tension estimation system, the natural frequencies of the cables were calculated from the raw acceleration time histories. A sample acceleration time history from cable 1 in group L7 (cable L7-1) is shown in Figure 5.17(a). The PSD was then determined using a Hanning window of 1024 data points with 50% overlap. The use of a Hanning window is more appropriate for random ambient excitations, providing a more dependable Fourier transformation of a signal because it begins and ends at a value of zero. With an increased number of data points, the natural frequencies were determined using the peak-picking method with higher precision. The Wolfram package for PSD calculation was run several times varying the standard deviation for Gaussian blur until the result was satisfactory for peak selection. Figure 5.17(b) shows the calculated PSD plot from cable L7-1, where the peaks with red markers represent the first four natural frequencies of the cable.

![Figure 5.17](image-url)

**Figure 5.17.** (a) Acceleration time history of cable L7-1; (b) power spectral density of cable L7-1.

The first four natural frequencies of each cable were determined from the PSD plots. These were used to calculate a linear model for each cable. The only exception to this was cables L3-3 and L3-4, which had a fourth frequency that exceeded the cutoff frequency, so the fourth modal frequency was not used to determine the linear fit. The linear fit for cable L7-1 is shown in Figure 5.18. The natural frequencies, fitted fundamental frequency, and coefficient of determination for each cable are shown in Table 5.5.
The correlation to a linear model was very strong since the coefficient of determination was close to 1.0 in all cables. The fitted frequencies were then used to calculate the estimated tension forces using Methods 1, 2, 3, and taut string theory in the Wolfram packages. All tensions were successfully determined by the tension estimating system. Table 5.6 summarizes the tension force estimations for each cable using all four methods compared to the original estimation using taut string theory. A profile of the cable tension forces is shown in Figure 5.19. This chart shows that the tension-estimating methods are in well agreement with the initial tension force estimations.

![Graph showing experimental natural frequencies and linear regression for cable L7-1.](image)

**Figure 5.18. Experimental natural frequencies and linear regression for cable L7-1.**

<table>
<thead>
<tr>
<th>Cable</th>
<th>$f_1$ (Hz)</th>
<th>$f_2$ (Hz)</th>
<th>$f_3$ (Hz)</th>
<th>$f_4$ (Hz)</th>
<th>$f_{fit}$ (Hz)</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>L3-1</td>
<td>9.236</td>
<td>18.81</td>
<td>29.37</td>
<td>41.64</td>
<td>10.05</td>
<td>0.991</td>
</tr>
<tr>
<td>L3-2</td>
<td>9.419</td>
<td>19.14</td>
<td>29.71</td>
<td>42.19</td>
<td>10.19</td>
<td>0.992</td>
</tr>
<tr>
<td>L3-3</td>
<td>11.43</td>
<td>23.76</td>
<td>38.04</td>
<td>-</td>
<td>12.36</td>
<td>0.992</td>
</tr>
<tr>
<td>L3-4</td>
<td>13.88</td>
<td>28.49</td>
<td>44.57</td>
<td>-</td>
<td>14.61</td>
<td>0.997</td>
</tr>
<tr>
<td>L4-1</td>
<td>6.657</td>
<td>13.40</td>
<td>20.32</td>
<td>27.74</td>
<td>6.846</td>
<td>0.999</td>
</tr>
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<td>12.81</td>
<td>19.50</td>
<td>26.53</td>
<td>6.554</td>
<td>0.999</td>
</tr>
<tr>
<td>L4-3</td>
<td>7.006</td>
<td>14.06</td>
<td>21.30</td>
<td>28.92</td>
<td>7.157</td>
<td>0.999</td>
</tr>
<tr>
<td>L4-4</td>
<td>7.688</td>
<td>15.59</td>
<td>24.01</td>
<td>32.60</td>
<td>8.043</td>
<td>0.998</td>
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</table>

**Table 5.5. Natural frequencies and fitted fundamental frequencies of Arrigoni Bridge cables.**
<table>
<thead>
<tr>
<th>Cable</th>
<th>( f_1 ) (Hz)</th>
<th>( f_2 ) (Hz)</th>
<th>( f_3 ) (Hz)</th>
<th>( f_4 ) (Hz)</th>
<th>( f_{1,fit} ) (Hz)</th>
<th>( R^2 )</th>
</tr>
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<tbody>
<tr>
<td>L5-1</td>
<td>5.292</td>
<td>10.72</td>
<td>16.19</td>
<td>21.97</td>
<td>5.439</td>
<td>0.999</td>
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<tr>
<td>L5-2</td>
<td>4.743</td>
<td>9.719</td>
<td>14.84</td>
<td>19.80</td>
<td>4.930</td>
<td>1.000</td>
</tr>
<tr>
<td>L5-3</td>
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<td>9.386</td>
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<td>4.769</td>
<td>0.999</td>
</tr>
<tr>
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<td>19.60</td>
<td>4.865</td>
<td>1.000</td>
</tr>
<tr>
<td>L6-1</td>
<td>4.212</td>
<td>8.424</td>
<td>12.77</td>
<td>17.11</td>
<td>4.260</td>
<td>1.000</td>
</tr>
<tr>
<td>L6-2</td>
<td>4.429</td>
<td>8.901</td>
<td>13.46</td>
<td>18.06</td>
<td>4.495</td>
<td>0.998</td>
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<td>L6-3</td>
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<tr>
<td>L7-3</td>
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<td>12.20</td>
<td>16.38</td>
<td>4.077</td>
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<tr>
<td>L7-4</td>
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<td>12.17</td>
<td>16.29</td>
<td>4.062</td>
<td>1.000</td>
</tr>
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<td>3.863</td>
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<td>3.256</td>
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<td>3.296</td>
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<td>L10-3</td>
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<td>15.39</td>
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<td>L10-4</td>
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<td>7.006</td>
<td>10.53</td>
<td>14.13</td>
<td>3.521</td>
<td>1.000</td>
</tr>
</tbody>
</table>

*Cables 3-3 and 3-4 displayed very high frequencies. Therefore, the fourth natural frequency was beyond of the cutoff frequency in the PSD plot and could not be determined.*

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Table 5.6. Tensions estimated for Arrigoni Bridge cables compared to original estimations.

<table>
<thead>
<tr>
<th>Cable</th>
<th>Taut String Theory</th>
<th>Method 1</th>
<th>Method 2</th>
<th>Method 3 (f₁ₙₜ)</th>
<th>Method 3 (f₄ₙₜ)</th>
<th>Original Estimation</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Force (kip)</td>
<td>% diff</td>
<td>Force (kip)</td>
<td>% diff</td>
<td>Force (kip)</td>
<td>% diff</td>
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<tr>
<td>L3-1</td>
<td>45.89</td>
<td>-6.09</td>
<td>36.15</td>
<td>-26.03</td>
<td>36.24</td>
<td>-25.84</td>
</tr>
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<td>L3-2</td>
<td>47.14</td>
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<td>37.27</td>
<td>-26.00</td>
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<td>-25.84</td>
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<tr>
<td>L3-3</td>
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<td>-19.71</td>
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<td>-19.81</td>
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<td>-16.72</td>
<td>82.71</td>
<td>-16.95</td>
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<tr>
<td>L4-1</td>
<td>47.91</td>
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<td>41.32</td>
<td>-16.08</td>
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<td>-16.34</td>
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<td>-14.59</td>
</tr>
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<td>-14.21</td>
<td>33.52</td>
<td>-14.51</td>
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<td>-15.22</td>
<td>34.97</td>
<td>-15.52</td>
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<tr>
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<td>38.24</td>
<td>-9.24</td>
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<td>41.50</td>
<td>-14.39</td>
</tr>
<tr>
<td>L6-3</td>
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<td>-11.31</td>
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<td>-11.64</td>
</tr>
<tr>
<td>Cable</td>
<td>Taut String Theory</td>
<td>Method 1</td>
<td>Method 2</td>
<td>Method 3 ($f_{1,\tilde{m}}$)</td>
<td>Method 3 ($f_{4,\tilde{m}}$)</td>
<td>Original Estimation (kip)</td>
</tr>
<tr>
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<td>-------------------</td>
<td>----------</td>
<td>----------</td>
<td>-----------------------------</td>
<td>-----------------------------</td>
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<td>43.34</td>
<td>46.73</td>
<td>44.92</td>
</tr>
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</table>

* Cables 3-3 and 3-4 displayed very high frequencies. Therefore, the fourth natural frequency was beyond of the cutoff frequency in the PSD plot and could not be determined.

† Original estimations for Cables 10-2 and 10-3 were not calculated. Thus, the percent differences of these estimations could not be calculated.
Cable group L2 could not yield tension estimation results because the ambient excitation was insufficient to excite its modal frequencies. Cables L10-2 and L10-3 had a ladder attached to them for access to the upper steel truss. This changed their lower natural frequencies significantly, so the tension force was not estimated for these cables in the previous study. As noted by the coefficients of determination in Table 5.5, the frequencies in cables L10-2 and L10-3 do not fit a linear model as perfectly as the other two cables in the same group. However, the correlation is strong and the estimated tensions are comparable to cables L10-1 and L10-4 using all methods.

As corroborated with the laboratory experiment, estimations determined by taut string theory and Method 3 when using the fundamental frequency are the most conservative, providing estimations close to those determined by the previous study. However, higher modes in Method 3 yield results closer to tension determined by Methods 1 and 2. According to the results determined in the laboratory, Methods 1

![Figure 5.19. Cable tension force profile.](image-url)
and 2 provide results closer to the real cable tension. This corroborates that Method 3 is more accurate when using higher frequency modes in cables with large $\xi$. By inspecting Figure 5.19, it is clear that the estimations are closer to one another when the cable is longer. Tension estimations determined with taut string theory in longer cables are close to those determined by Methods 1 and 2. This corroborates that taut string theory is best applicable to longer cables that more closely resemble a string with very low bending stiffness. This applies to Method 3 as well, since the differences between estimations using different natural frequencies are smaller in longer cables and more apparent in shorter cables, implying that the bending stiffness contribution becomes less relevant in longer cables (as implied in Equation (5.5)). These results provide an assessment of the loading condition of the critical load-carrying members that calls for a careful inspection to determine the source of excessive tension load in cables L3-3, L3-4, and L4-4. The tension estimation system successfully performed all spectral analysis and tension estimation calculations for field data, providing valuable information about the loading condition of the suspension cables in the Arrigoni Bridge.

5.6 Conclusions

A wireless cable tension estimating system has been successfully developed and validated via a laboratory-scale experiment and field data from a truss arch suspension bridge. These experiments allowed to further verify the effectiveness of the onboard algorithms and to compare the tension estimation methods. It was found that Methods 1 and 2 are the most accurate because they consider bending stiffness and use empirically determined coefficients in their formulations. Method 3 is conservative when using the fundamental frequency, but becomes more accurate with higher-mode frequencies in the examined cables. Similarly, taut string theory offers conservative tension estimations. However, when longer cables are used, estimations calculated with taut string theory or Method 3 become more accurate. This is so because the assumption of an ideal taut string becomes true in longer cables and the role of bending stiffness becomes less relevant. The validation performed using field-data also
allowed the users to identify suspension cables that were excessively loaded, demonstrating the relevance of performing regular cable tension load condition assessment.

Using a single-board microcomputer and peripheral sensors, the wireless tension estimating system effectively collects acceleration time history from a field-ready USB accelerometer, calculates PSD, generates ready-to-view PSD and acceleration time history plots, and selects natural frequencies from the PSD plot automatically using the peak-picking method. It also allows user inputs for accelerometer settings, PSD parameters, cable properties, and final natural frequency selection for quality assurance. The system also calculates a linear model for natural frequency fitting using the final selected natural frequencies and calculates cable tension using four different vibration-based tension estimating methods. All these actions can be performed remotely, a great convenience for field usage, since it can be operated from a remote computer via a WLAN. This wireless system can further be expanded to incorporate additional monitoring features, such as vehicle counting, a weather station, and further SHM vibration-based methods. The development of this system poses a relevant advancement in the practical usage of low-cost and simple vibration-based technology for reliable SHM.
Chapter 6 Development and Experimental Assessment of an RFID-based Crack Sensor for Metallic Structures

6.1 Introduction

The timely detection of fatigue crack formation in many engineering applications such as railways, bridges, aircraft, and pipelines is crucial for safety, extended service life, and reduced maintenance cost. Cyclic loading can produce fatigue, which occurs progressively and locally, leading to sudden failure below the yield stress limit. Traditionally, highway bridges use various types of steel-welded beams that are susceptible to these effects. For decades, cover plates were added to the flanges of such beams to increase their flexural capacity [117]. The use of cover plates in regions of high applied moment allows sections on primary beams to be lighter while providing more flexural capacity [115,116]. However, the use of welded cover plates introduces a transverse weld periphery at the toe, forming a line of elevated tension where fatigue cracks can form prematurely and propagate into the main beam. The National Cooperative Highway Research Program [4] noted that fatigue crack propagation at nearly all other structural details occurred as cracks initiated from the toe of fillet or groove welds because of high stress concentration due to discontinuity and residual tension stress. Thus, welded cover plated beams are more susceptible to fatigue crack formation than rolled beams, and this requires closely monitored information of stress level by frequent visual inspection and field measurement.

The Federal Highway Administration has mandated that all highway bridges located on public roads be inspected every two years [113]. Most inspections rely exclusively on visual means. Crack monitoring activities using crack-detecting sensors can complement visual inspections and provide information about crack damage. As such, extensive research has been performed in flaw-detecting sensors and many non-destructive evaluation and testing methods have been developed.

A few methods to detect cracks in metallic materials have been developed. One of the most common and effective methods to detect flaws and cracks is using angle beam ultrasound [119]. The
ultrasonic wave can be generated by wireless inductively-coupled piezoelectric transducers. This wave is reflected back to the transducer by some form of discontinuity, such as another surface or a crack. Typically, crack inspection with this method requires an experienced operator and it is manually performed by moving the transducer across the surface at different orientations. Another mature technology is the usage of eddy currents where the alternation of the electromagnetic field generates magnetic lines of force that reveal hidden defects. This method is very effective in determining subsurface cracks due to its great penetration depth [120,121]. Other technologies that have been patented or are commercially available include ultrasonic flaw detectors [122], magnetorestive sensors [123], surface-mount piezoelectric paint sensors [124,125], probe-pump-based Brillouin sensor systems [126], coaxial cable sensors [127], and fiber-optic sensors [128], among others. However, the use of these methods for long-term monitoring of crack patterns in larger scale civil infrastructure is time-consuming and expensive.

Passive radio frequency identification (RFID) antennas have been studied to provide a low-cost method for crack detection. Kalansuriya et al. [8] introduced the concept of using RFID tag antennas to sense surface cracks and the usage of a grid of RFID tags to monitor crack patterns in civil infrastructure. The application of RFID technology for crack monitoring relies on the permanent changes in impedance and radiation efficiency caused by the presence of a crack. The RFID method uses electromagnetic transmission by means of a radio frequency-compatible integrated circuit to retrieve data from tags and send it to reader antennas. The communication protocols between the RFID tag and the reader are standardized and efficient, making RFID an ideal wireless communication infrastructure for dense arrays of sensors. The greatest advantage of RFID technology is the elimination of coaxial cables and the reduction of installation time and maintenance costs. Passive UHF RFID dipole tags are also very inexpensive, ranging $0.10-$0.20 per unit in mass production [132]. Passive RFID tags also possess the advantage of not depending on an external power source for operation, one of the most recurrent and concerning problems in non-destructive evaluation and health monitoring applications.
Kalansuriya et al. [8] presented a method where the tag antenna is sensitive to cracking for 50% of the radiating element’s length. To counteract this limitation, they used 2D arrays. However, these grids contained gaps where crack detection would be missed and need to be optimized [8,207]. In addition, Kalansuriya et al. [8], Mohammad and Huang [9], and Cazeca et al. [10] developed in-house RFID antenna-based sensors for crack detection using their knowledge as electrical engineers. From a civil engineering standpoint, the assembly of an RFID-based sensor is costly given the typical large deployments needed. Commercially available RFID technology is more viable due to the reduced unit cost in comparison to in-house developments. However, the performance of low-cost commercial RFID tags for crack detection has not been studied. In consequence, significant research is needed to determine the sensitivity to damage of commercial RFID technology and to establish a guideline for deployment regarding preferred orientation, placement, and configuration for reasonable results.

This chapter presents an initial study on damage sensitivity of ultra-high frequency (UHF) passive RFID tags for crack detection on steel structures. Damage sensitivity has been determined to be the ability of the tag to return a different backscatter power signal upon changes to the underlying metallic surface and changes to its own integrity. A crack detection sensing system has been developed for two cases: using a single RFID tag and using a 2D array of RFID tags lain over a substrate material. Its performance has been validated by laboratory-scale experiments. These experiments explore the effects on backscatter power caused by: (1) read distance for tag antennas severed in difference locations, (2) damage stages as a crack propagates from an underlying metallic surface into the substrate and into a single tag, and (3) damage stages as a crack propagates from an underlying metallic surface into the substrate and into a tag in a 2D array. The system overcomes the adverse effect of the metallic structure on the backscatter signal of the RFID tag while maintaining a high tag density of the array for increased sensing pervasiveness.
6.2 Theoretical background

6.2.1 Backscatter power

Kalansuriya et al. [8] identify backscatter power as the most important measured value from a tag for crack detection and characterization. Backscatter power can be described by the radar range equation:

\[ P_R = P_T G_T G_R \left( \frac{\lambda}{4\pi R_T R_R} \right)^2 \sigma \]  \hspace{1cm} (6.1)

where \( P_R \) is the backscatter power, \( P_T \) is the transmitted power, \( G_T \) is the transmitting antenna gain, \( G_R \) is the receiving antenna gain, \( \lambda \) is the signal wavelength, \( R_T \) is the distance between the target (the tag chip, in this application) and the transmitting antenna, \( R_R \) is the distance between the target and the receiving antenna, and \( \sigma \) is the target’s radar cross section. In a monostatic scattering application, the antenna emitting the electromagnetic signal also receives the echo from the target tag chip so that \( R_R = R_T \) and \( G_R = G_T \).

Equation (6.1) shows that backscatter power is attenuated as the reading distance increases by a quartic factor. \( P_R \) can be determined from the received signal strength indicator (RSSI) logged by the reader equipment from the following expression:

\[ RSSI = 10 \log_{10} \left( \frac{P_R}{1\ mW} \right) \]  \hspace{1cm} (6.2)

RSSI is then a decibel expression (dBm) of backscatter power.

Most reading equipment uses frequency hopping spread spectrum (FHSS), so the value of RSSI depends on the transmit frequency channel. The signal frequency varies in the range of 902-928 MHz in North America. The Impnj Speedway MultiReader software used in this study performs a frequency hopping sequence by changing the transmitting channel during each inventory session. Table 6.1 shows the signal frequency assigned to each channel. This study uses average values of RSSI of data samples equally weighted across all frequency channels.
Table 6.1. Impnj Speedway reader frequency plan for North America.

<table>
<thead>
<tr>
<th>Channel number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>…</th>
<th>49</th>
<th>50</th>
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<tr>
<td>Center Frequency (MHz)</td>
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<td>903.25</td>
<td>903.75</td>
<td>904.25</td>
<td>…</td>
<td>926.75</td>
<td>927.25</td>
</tr>
</tbody>
</table>

6.2.2 Substrate material: Image theory behind RFID wave propagation

The usage of RFID technology for crack detection on metallic surfaces adds a layer of complexity to the received backscatter power signal related to image theory. A metallic plate is a large conductive surface that behaves as a ground plane. The backscatter power signal received at the antenna is equal to:

\[
\exp(-j\beta_{\text{air}}z) \left( I_{\text{tag}} + I_{\text{ground}} \exp(-j\beta_{\text{material}}\Delta z) \right) \tag{6.3}
\]

where, \( \beta_{\text{air}} \) is the phase constant of air, \( \beta_{\text{material}} \) is the phase constant of the substrate material (a material separating the tag from the metallic surface), \( I_{\text{tag}} \) is the primary backscatter of the tag, and \( I_{\text{ground}} \) is the image backscatter. For a horizontal tag (i.e. a tag oriented parallel to the ground plane), \( I_{\text{tag}} = - I_{\text{ground}} \). If a horizontal metallic RFID tag is placed directly on the metallic surface acting as a ground plane (\( \Delta z = 0 \)), the power signal received at the antenna will be zero.

The magnitude of the reflected wave can be maximized when the reflection of the RFID tag comes from an image source located half a wavelength below (see Figure 6.1) since a half wavelength corresponds to a 180° phase change. In practical terms, this means that, ideally, the RFID tag should be placed at a quarter wavelength in front of the metallic surface in order to maximize the power received by the antenna.
The antenna used in this investigation is a right hand circularly polarized antenna that transmits signals at frequencies ranging from 902-928 MHz. Since the wavelength of these signals is approximately 1 ft, a substrate material closely resembling the relative permittivity of air (such as polystyrene foam) should ideally be 3 in thick. However, such a thick substrate would raise concerns on the structural effect that it would sustain on the girder. If the material selected to separate the tag from the metallic surface has the adequate dielectric properties, it can be used to reduce the physical separation between the tag and the ground plane. The wavelength of an electromagnetic wave in a dielectric medium is given as:

\[ \lambda = \frac{c}{f} \cdot \frac{1}{\sqrt{\epsilon_r}} \]  

(6.4)

where, \( f \) is the wave frequency, \( c \) is the speed of light, and \( \epsilon_r \) is the relative permittivity of the material between the target tag and the ground plane. In vacuum conditions, \( \epsilon_r = 1 \). The relative permittivity of air is approximately 1. Relative permittivity can be related to the phase constant by:

\[ \epsilon_r = \left( \frac{\beta c}{2\pi f} \right)^2 \]

(6.5)

According to Equation (6.4), a material with higher permittivity also reduces the wavelength in a dielectric medium, thus reducing the required distance \( \Delta z \) to maximize the received backscatter power.
Therefore, a thinner substrate made of a material with a relative permittivity greater than one is preferred to increase the received backscatter power signal. This substrate must also be sufficiently elastic in order to transfer the strains at the extreme tension fiber of the girder into the RFID tag and it should also be able to adhere to a metallic surface with weatherproof adhesive. From the available materials that fulfill all of these characteristics, ethyl-vinyl acetate (EVA) rubber, which has a relative permittivity of approximately 2.8, has been found to increase RSSI satisfactorily while also being a very flexible and durable material.

6.3 Single sensor development

6.3.1 Single RFID-based crack sensor configuration

The components of a single crack detection sensor include a commercial RFID tag, a layer of a substrate material, and adhesives (see Figure 6.2). Multiple adhesives including epoxy, double-sided tape, and a cyanoacrylate-based glue were used to test their effect on RSSI and no significant difference was found. Epoxy was finally chosen as an adhesive for bonding at both interfaces in field deployment because of its proven effectiveness in strain transferring applications, such as with optical fiber sensors.

Figure 6.2. Single sensor configuration of RFID crack sensor.

The commercial UHF RFID tag used in this study is the Alien Technology ALN-9662 Short Inlay tag. This tag is EPG Gen 2 and ISO/IEC 18000-6C compliant and it uses a Higgs 3 EPC Class 1 Gen 2 RFID tag integrated circuit (IC). The tag antenna is made of a flexible metallic material, which is adhered to a wet inlay.

The substrates used in these initial tests are EVA rubber or polystyrene foam. Polystyrene foam is used for experiments where a material with relative permittivity similar to that of air is desired for basic evaluations of backscatter power behavior. Since EVA rubber has a higher relative permittivity and is
flexible and durable, it is used in experiments for sensing system performance evaluation where higher RSSI readings are desired.

### 6.3.2 Performance evaluation experiments

There are two directions of fatigue crack propagation that are possible in beams with welded cover plates: (1) crack propagation from the beam into the substrate material and then into the tag, causing the tag antenna to be severed and (2) propagation along the beam surface beneath the sensing system. The first case implies that the substrate material and eventually the tag antenna will be severed. The second case assumes that the separation of the metallic material will induce strain into the substrate material and thus into the tag antenna. The direction of crack propagation assumed in the initial study presented in this chapter is the first case. For the effects of the second case, sensitivity to crack propagation was tested and is presented in Chapter 7.

#### 6.3.2.1 Read distance and loss of the effective area of a tag antenna.

As implied by Equation (6.1), greater read distances cause backscatter power from the tag in question to be reduced by a quartic factor. The effects of read distance on RSSI along with the effect of severing a portion of the tag antenna have been studied. RSSI was measured for read distances at 3-foot intervals up to 15 ft, as shown in Figure 6.3. The RFID tag was attached to a polystyrene foam block. Polystyrene foam is known to have low reflective and absorptive properties, closely simulating air.
The subsequent measurements of RSSI were made for tags that were severed at different locations, simulating different scenarios of an underlying crack that has propagated into the antenna. The two cutting sequences shown in Figure 6.4 were examined. Each cut number represents a different location in which a crack disconnects an additional portion of the antenna from the IC. For example, the damage scenario #2 in sequence #1 represents the situation in which a crack along the dotted line #1 and another crack along the dotted line #2 disconnected the portions to the right of the dotted line #1 and to the left of the dotted line #2. Scenario #3 represents two cracks that disconnected the portions to the right of dotted line #3 and to the left of dotted line #2. The RSSI at each stage in the cutting sequence was measured at the distances shown in Figure 6.3.
The trends of RSSI versus damage scenario and read distance for both sequences are shown in Figure 6.5. As greater areas of the tag antenna were disconnected from the tag IC, RSSI exponentially decayed. Cuts made between the IC and the square patches do not yield as great a gradient in RSSI as cuts made within the square patches. When greater antenna surface area was disconnected from the IC loop, a larger drop in RSSI was detected. Therefore, damage within the tag can be more precisely located when the damage is within the square patch than when the damage is between the patch and the IC.

The reader was incapable of detecting the tag backscatter power at further distances. The dependence of RSSI on read distance establishes a requirement for read distance standardization. Since RSSI decreases with distance, the chosen parameter for crack detection will depend on the final choice of read distance.

Finally, it was noted that RFID tags could not reflect power when receiving direct sunlight. This limits the usage of RFID tags for crack detection to shaded areas. As the bridge deck covers bridge girders throughout most of the day, it is anticipated that proper scheduling will suffice for proper damage identification in the field.

![3D Plot of ALN-9662 Tag (Configuration 1)](image1)

![3D Plot of ALN-9662 Tag (Configuration 2)](image2)

**Figure 6.5.** RSSI vs. damage scenario and read distance: (a) sequence #1, (b) sequence #2.
6.3.2.2 Damage detection of a crack propagating into a tag antenna. A second set of tests was conducted to demonstrate the crack propagation detection capability of commercial RFID tags on a metallic surface. Figure 6.6 shows the overall arrangement for these experiments. A PC with Impnj MultiReader software was connected to an Impnj Speedway Revolution R420 UHF RFID Reader. A high gain circular right hand polarized patch antenna was connected to the reader. A 1/8 in thick rectangular aluminum plate was used as a test specimen. A 1 in long incision was made into one side with a 1/16 in vertical band saw blade. The opposing side was left unaltered as a control surface.

As expected, no radiation was detected from the tag when it was directly attached to the metallic surface. A polystyrene foam plate was used as a substrate material to simulate air as closely as possible. In a similar setup, an EVA foam plate was also used as a substrate material for comparison. The aluminum plate, polystyrene foam or EVA foam plate, and the ALN-9662 RFID tag were raised onto a stack of polystyrene foam so that the tag IC would be elevated to the same height as the center of the reader antenna while avoiding excessive radiation interference. The read distance was fixed at 3 ft.

Four damage scenarios were tested for detection: (1) undamaged surface with uncut tag, (2) cracked surface with uncut tag, (3) cracked surface and cracked substrate material with uncut tag, and (4) cracked surface with cut tag (see Table 6.2). Each scenario represents a stage in crack formation and propagation, the first being no damage at all and the last being the ultimate damage case. Damage stage #2 shows the initial stage of crack formation on the metallic surface while the sensing system is untouched. The inclusion of damage stage #3 ensured to account for the changes in system impedance that are to be expected upon the comprisal of a substrate material with a higher permittivity, as in the case of EVA foam. Since polystyrene foam has a permittivity close to that of air, the damage scenarios that were tested when this material was the substrate were damage scenarios #1, #2, and #4. Figure 6.7 shows the tag placement on the substrate materials and the metallic surface.

It is not anticipated that EVA foam will break under field conditions, particularly with minor cracking. However, it is expected that damage stage #2 will occur in the field and that cracks will propagate parallel to the monitored surface. The inclusion of damage stages #3 and #4 in the present study
serves the purpose of providing a complete picture of tag sensitivity to damage propagating perpendicularly to the monitored surface.

Figure 6.6. Setup for crack detection tests.

Table 6.2. Conditions in each damage scenario.

<table>
<thead>
<tr>
<th>Damage scenario</th>
<th>Sensing component</th>
<th>Metallic surface</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Tag</td>
<td>Substrate</td>
</tr>
<tr>
<td>1</td>
<td>Intact</td>
<td>Intact</td>
</tr>
<tr>
<td>2</td>
<td>Intact</td>
<td>Intact</td>
</tr>
<tr>
<td>3</td>
<td>Intact</td>
<td>Crack damage</td>
</tr>
<tr>
<td>4</td>
<td>Crack damage</td>
<td>Crack damage</td>
</tr>
</tbody>
</table>
Figure 6.7. Front view of aluminum plate, foam plate, and RFID tag: (a) 0.4290 in thick polystyrene foam, (b) 0.5 in thick EVA foam.

Figure 6.8 shows the resulting measurements of RSSI for all damage scenarios using intermediate polystyrene foam and EVA foam. The overall performance of the tag was improved (i.e. RSSI was increased) when EVA foam used, as raw RSSI ranged between -52.87 and -40.46 dBm compared to the range of -56.03 and -45.38 dBm when polystyrene foam was used. As a crack formed on the metallic surface and propagated up to the substrate, RSSI consistently increased regardless of the substrate material. A drop in RSSI occurred upon the cracking of the tag antenna (damage scenario #4) compared to the control (damage scenario #1) when either EVA or polystyrene foams were used. It is clear then that the underlying metallic surface increases the radiation efficiency of the system when a crack is present on the left side of the tag IC with respect to the direction of the incident electromagnetic wave. It is therefore possible to detect an underlying crack that has not propagated into the RFID sensor and that has opened a gap across the depth of the metallic surface.
Figure 6.8. RSSI for crack detection experiments: (a) 0.4290 in thick polystyrene foam, (b) 0.5 in thick EVA foam.

6.4 2D sensor array development

6.4.1 Multiple sensor array configurations

To increase the pervasiveness of the crack propagation monitoring system, 2D arrays of tags were considered. It is known that the proximity of RFID tags has an effect in their sensitivity, causing some tags to report a gain or a reduction in backscatter power depending on the layout of the surrounding tags. This change in sensitivity is caused by tag detuning, tag shadowing, and re-radiation cancelation, collectively known as coupling or proximity effects [208].

A 2D array of tags should behave in a way analogous to the parasitic elements in a Yagi-Uda antenna [209]. The principal tag of interest in an array (hereafter referred to as the control tag) would be the driven element. The RSSI of this tag will be the principal indicator of crack formation and propagation. The strength of the RSSI of the control tag will be influenced by the surrounding tags in the array, similar to how director parasitic elements work together in the Yagi-Uda antenna to increase the antenna’s gain. Therefore, a 2D array must be selected such to improve pervasiveness with close spacing and to enhance sensitivity to damage with placements that increase RSSI in the control tag. Different tag array configurations were studied based on their effects on the RSSI of a control tag. There are two
features that are primarily pertinent to the development of the tag array to be used in this sensing system: spacing between tags and configuration.

6.4.1.1 Spacing between tags in 2D arrangement. Coupling effects between tags can either increase or decrease the backscatter power of the tags involved. In order to optimize the backscatter power of a control tag (T-1) the configurations shown in Table 6.3 were tested for the following separations: 1/8, 1/4, 1/2, 1, and 2 in. Distance 1-2 refers to the separation between tags T-1 and T-2 and distance 1-3 refers to the separation between tags T-1 and T-3. Tag separation was measured as a clear distance from the edge of a patch of one tag to the nearest edge of the patch of the other tag. Figure 6.9 shows the setup used for all tag array experiments. The substrate material used in this experiment to separate the array from the 1/8 in thick aluminum plate was polystyrene foam and the read distance was kept constant at 3 ft in reference to the control tag, T-1.

Table 6.3. RFID tag configurations used for spacing optimization.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4A</th>
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<tbody>
<tr>
<td></td>
<td>![T-1]</td>
<td>![T-1]</td>
<td>![T-2]</td>
<td>![T-1]</td>
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<td>![T-3]</td>
<td>![T-3]</td>
<td></td>
<td>![T-3]</td>
</tr>
</tbody>
</table>

Figure 6.9. Experimental setup for 2D array tag experiments.
Figure 6.10 and Figure 6.11 show the variation in RSSI on tag T-1 for different separations. The blue filled-in circle indicates the original received power of T-1 in standalone configuration (C1). Figure 6.12 shows the values of RSSI for all spacing combinations in configuration C4A. The spacing in horizontal configuration (C2) improved RSSI the most in tag T-1 at 1/4 in separation. The spacing in vertical configuration (C3) improved RSSI the most in tag T-1 at 1/8 in separation. Figure 6.12 shows that the same spacing increases RSSI the most. Therefore, the spacing combination that best increases RSSI is 1/4 in for horizontal spacing and 1/8 in for vertical spacing. This spacing will be used in all experimentation involving 2D arrays.

Figure 6.10. RSSI of T-1 in configuration C2.

Figure 6.11. RSSI of T-1 in configuration C3.
6.4.1.1 Array placement. The position of a tag in relation to others can also have a significant impact on its backscatter power. Since crack propagation monitoring requires an increased number of sensing units for greater pervasiveness, a basic array of 3 rows by 2 columns of RFID tags was chosen to determine the best configuration for increased received power. Using polystyrene foam as the substrate material, the configurations shown in Table 6.4 were tested for RSSI in the control tag, T-1. The array was placed on a 1/8 in thick aluminum plate. These configurations have been ordered from the one yielding the lowest RSSI in T-1 to the one yielding the highest RSSI in T-1. Figure 6.13 shows a bar graph of the ordered configurations.

Figure 6.12. RSSI of T-1 in configuration C4A.

Figure 6.13. 2D array configurations sorted by RSSI on T-1.
Table 6.4. 2D array configurations sorted by RSSI on T-1.

<table>
<thead>
<tr>
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<th>C7D</th>
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<table>
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</table>
The configuration that most reduced backscatter power was a vertical array (C3) and the configuration that most increased backscatter power was a purely horizontal one (C2). The RSSI of T-1 was generally improved (in reference to itself in standalone configuration – C1) when there was at least one other tag somewhere on the column next to it. This implies that the array should include tags side by side to the control tag. This also suggests that a combination of configurations C4A, C4B, C4C, C5A, C5B, and C2 will be best to maintain a high RSSI while increasing pervasiveness. To maintain consistency and cover as much surface area as possible, configurations C5A and C5B, i.e. a 2 by 2 configuration with 4 tags, were preferred for the performance evaluation of the array.

6.4.2 Performance evaluation experiment: 2D array

Using the same setup as in previous 2D array experiments (see Figure 6.9), a 2D array placed on a 0.5 in thick EVA foam sheet conforming to configuration C5A was used to verify the sensitivity of the system (see Figure 6.14). The same four damage scenarios explained in Table 6.2 were examined: (1) undamaged surface with uncut tag, (2) cracked surface with uncut tag, (3) cracked surface and substrate with uncut tag, and (4) cracked surface, substrate, and tag. Figure 6.15 shows the final damage stage (damage scenario #4). The crack made to the aluminum plate was a 1 in long incision into one side with a 1/16 in vertical band saw blade. The average RSSI of the control tag (upper left tag in the array) was used to compare changes in backscatter power at each damage stage.
Figure 6.14. 2D array employed in the experiment: (a) front view of configuration C5A, (b) top view of sensing system; from bottom to top: metallic surface, EVA foam, RFID tag.

Figure 6.15. Damage stage #4 of 2D array: (a) front view, (b) top view.

The pattern of RSSI changes in the control tag as a crack propagated into the system is shown in Figure 6.16. The selection of the 2D array configuration C5A lain on EVA foam and oriented perpendicular to the reader antenna showed an improvement in performance, yielding RSSI values ranging between -42.91 and -39.25 dBm. In this 2D array, damage to the metallic surface underneath the
right side of the control tag IC caused a small drop in RSSI. Further damage in the substrate decreased RSSI slightly more. Finally, the ultimate damage state (damage scenario #4) increased RSSI significantly, providing a notable change to indicate damage propagation onto the tag antenna. This trend is mirrored to the pattern observed in the single tag system because the location of the crack in the 2D array performance evaluation experiment was on the right side of the IC with respect to the incident electromagnetic wave instead of on the left side of the IC as was the case in the single tag performance evaluation experiment. This behavior has been observed in in-house developments of linearly polarized RFID-based crack sensors where the direction of the change in backscatter power depends on the location of the crack with respect to the IC [8].

![Figure 6.16. RSSI of control tag in a 2D array for damage scenarios 1-4.](image)

### 6.5 Damage index

Performance evaluation studies have revealed that raw RSSI varies from one commercial RFID tag to another when all other experimental parameters remain unaltered. For further automation of crack detection in bridges, establishing a damage index based on the change of RSSI instead of on raw RSSI values is desirable. Because the change in RSSI is proportional to the severity of crack propagation, percentage change in RSSI before and after crack damage can be used effectively to this end. The RSSI percentage change is:
\[ \Delta \text{RSSI} \, (\%) = \left( \frac{\text{RSSI}_{\text{intact}} - \text{RSSI}_{\text{damaged}}}{\text{RSSI}_{\text{intact}}} \right) \times 100 \]  

where \( \text{RSSI}_{\text{intact}} \) is the RSSI of the unaltered state and \( \text{RSSI}_{\text{damaged}} \) is the RSSI of the damage stage in question.

The algebraic sign of the percentage change is an indicator of the location of the crack formation with respect to the tag IC. Figure 6.17 shows the location of the crack formation with respect to the tag IC viewed from the angle of the incident electromagnetic wave in all performance evaluation experiments. In single tag configuration experiments (Figure 6.17a and 6.17b), the crack was to the left of the IC while in the 2D tag array experiment (Figure 6.17c), the crack was to the right of the IC. Figure 6.18 and Figure 6.19 show the percentage changes in RSSI for the single tag configuration and the 2D array configuration, respectively.

![Figure 6.17](image)

**Figure 6.17.** Location of crack formation as viewed from the reader antenna: (a) single tag on polystyrene foam, (b) single tag on EVA foam, (c) 2D tag array on EVA foam.

Damage occurring to the left of the tag IC presented small increases in RSSI during initial damage stages. When damage was to the right of the tag IC, small decreases in RSSI were observed during initial damage stages. In the single tag performance evaluation experiment involving EVA, initial damage stages (scenarios #2 and #3) induced percentage changes in RSSI ranging from 3.097-3.586%. The same damage stages yielded an RSSI percentage change of 2.665% on a single tag configuration on polystyrene foam and a range of 1.586-2.52% on the control tag of a 2D array on EVA foam. Therefore, initial stages of damage can be detected when changes in RSSI are between 1.5% and 3.6%.

Furthermore, damage on the left of the tag IC yielded large drops in RSSI at the ultimate damage
state (scenario #4) while ultimate damage on the right of the tag IC exhibited the opposite behavior. Ultimate damage on a single tag configuration on EVA foam presented an RSSI percentage change of 26.00%. The same configuration and damage state on polystyrene foam yielded a change of 20.19%. Ultimate damage on the control tag of the 2D array on EVA foam caused a 6.215% reduction in RSSI. Thus, advanced damage stages will cause the direction of RSSI gradient to change with values greater than 6%.

![Graphs showing percentage change in RSSI](image)

**Figure 6.18.** Percentage change in RSSI of a single tag with respect to the undamaged condition: (a) 0.4290 in thick polystyrene foam, (b) 0.5 in thick EVA foam.

![Graph showing percentage change in RSSI for control tag](image)

**Figure 6.19.** Percentage change in RSSI of the control tag in a 2D array with respect the undamaged condition.
In summary, single tag and multiple tag configurations can both be used for specific crack monitoring situations. As percentage changes in RSSI in single tag configurations are larger, small cracks can be monitored more accurately. On the other hand, multiple tag arrangements reveal lower percentage changes in RSSI due to coupling effects. However, these gradients are sufficiently large to detect changes in longer cracks, which would produce larger percentage changes in RSSI. 2D arrays can also cover larger areas for expanded pervasiveness. The behavior of the sensing system makes percentage change in RSSI a competent damage index for crack monitoring.

6.6 Conclusions
An RFID-based crack sensor was successfully developed using low-cost commercial tags and its performance was validated with comprehensive laboratory-scale experiments. The percentage change in backscatter power from the RFID tags is a relevant damage index, as it is sensitive to crack propagation. The magnitude of the damage index consistently increases as crack damage gradually propagates from the metallic specimen to the substrate and the tag; this demonstrates the potential of the developed sensors for crack detection. Furthermore, the algebraic sign of the damage index indicates the location of damage within a tag with respect to its IC. A crack sensor with a single tag configuration displayed the best performance in crack detection with a high damage index change. Yet, a 2D array of tags is preferred to increase the sensing region. Guidelines on optimal spacing and configuration of multiple tags for the development and operation of RFID-based crack sensor arrays were defined. RFID crack sensor arrays are also effective at crack propagation monitoring, as indicated by the significant damage index changes measured in laboratory experimentation. Therefore, the developed system significantly propels the advancement of crack propagation monitoring on steel structures at a low cost, enabling extensive usage on welded cover plates within a restricted budget.
Chapter 7 Experimental Evaluation of a Low-cost RFID-based Sensor to Crack Propagation

7.1 Introduction

The USA faces a great challenge with bridge inspection for transportation safety. It has been determined that 9.1% of bridges are structurally deficient and more than half are reaching their design life [1]. In order to monitor the performance of said bridges, the Federal Highway Administration stipulates that all bridges on public roads be inspected every two years [113]. However, the reliability and frequency of inspection methods has room for improvement as structural health monitoring and nondestructive evaluation methods continue to become more informative and practical.

Some of the most failure susceptible bridge structures are steel bridges. Steel bridges account for more than 43% of substandard bridges in the USA and can deteriorate due to corrosion, increase in traffic volume, and deicing salts [114]. In addition, cyclic loading can produce local failure due to fatigue [12]. Traditionally, highway bridges use various types of connections with high stress concentration regions, which are particularly susceptible to these effects. For instance, the use of cover plates in regions of high applied moment allows sections on primary beams to be lighter while providing more flexural capacity [115,116]. However, weld defects in cover plates introduce high stress concentrations where fatigue cracks typically initiate and propagate, reducing the load capacity of the girders [117]. As another example, floor-beams are connected to main trusses by fabricating a gap that reduces the width of the upper flange of the floor-beam right at the connection, making the beam susceptible to cracks due to out-of-plane bending and due to secondary bending moment in the plane of the floor-beam web. Other crack-prone details include diaphragms and cross-bracing connections, copied and cut-short beam ends, and stringer-to-floor beam connections. The propagation of said cracks can significantly reduce the service life of steel bridges from their original design [118]. Considering the increased traffic volume and the large percentage of steel bridges in a bridge population with high incidence of structural deficiency
determined primarily by visual inspection carried out every 2 years, a system that can determine crack formation at an early stage is essential.

Many crack detecting and characterizing methods for metallic surfaces have been developed to supply this need. A common technology for flaw detection is angle beam ultrasound [119]. Another mature technology is the usage of eddy currents which reveals hidden defects at great penetration depth [120,121]. Other technologies include ultrasonic flaw detectors [122], magnetoresistance sensors [123], surface-mount piezoelectric paint sensors [124,125], probe-pump-based Brillouin sensor systems [126], coaxial cable sensors [127], fiber-optic sensors [128], acoustic emission technology [129], and large-area sensors [130,131]. Unfortunately, the use of these methods for long-term monitoring of crack patterns in larger scale civil infrastructure is time-consuming, expensive, and requires experienced operators.

Passive ultra-high frequency (UHF) radio frequency identification (RFID) antenna tags have been used in recent years to account for the costs and complexities of more established methods. RFID antennas undergo permanent changes in impedance and radiation efficiency upon changes to their surrounding electromagnetic environment, such as the presence of a crack on a metallic surface [12]. The wireless communication protocol between tag and reader is well established and reliable. Passive UHF RFID dipole tags are also very inexpensive, ranging $0.10-$0.20 per unit in mass production [132]. RFID-based sensors can also be easily assembled and installed. The wireless nature of this system along with its power independence and low cost allows it to potentially be deployed as smart dust to increase pervasiveness. Moreover, interrogation of several antenna sensors can be performed simultaneously, so that tag arrays can provide further information on crack characteristics and pervasively monitor a large-scale structure.

Several studies have been implemented for the development of antenna sensors for crack detection with potential application to civil structures. One such project is an in-house developed wired patch antenna [9] that experimentally demonstrated the ability to determine crack length and orientation by measuring resonant frequency shifts on a conductive surface. Another successful development [8] is a dipole antenna constructed with conductive paint and a copper antenna loop that was tested on previously
cracked reinforced concrete beams. In addition, a strain and crack detecting folded patch antenna was developed to monitor conductive surfaces [11].

Although all developments so far have been able to detect crack propagation in specific types of medium, none have combined 5 critical factors that any crack sensor ought to have in order to monitor steel bridge girders: (1) low cost, (2) wireless monitoring, (3) simple damage feature extraction, (4) sensitivity to crack presence on conductive surfaces, and (5) sensitivity to crack propagation on said surfaces. In order to monitor cracks propagating from areas of high stress concentration in large metallic structures using RFID technology, it is necessary to turn to lower cost, wireless options. Commercially available passive UHF RFID antenna tags can provide a solution to this problem, but have received reduced attention in comparison to in-house developments. Also, most successful developments use a vector network analyzer, which introduces an additional step of complexity in feature extraction to field users during inspection. In addition, none of the previous studies has verified sensor performance in face of a progressively propagating crack in a metallic medium with high precision measurements and ensuring small scale yielding, which is characteristic of in-service structures.

A wireless, passive UHF RFID crack sensor using commercial RFID tags was developed, as presented in Chapter 6 [12]. The damage index is based on direct backscatter power measurements that require minimal post-processing. As shown in Chapter 6, the crack sensor demonstrated to be capable of detecting large cracks on a metallic surface, specifically 1 mm wide and 8 mm long, and cracks that had compromised the integrity of the sensor antenna. In addition, a general locality of the crack could be determined based on the algebraic signal of the damage index. Single- and multiple-sensor configurations were tested to determine the best configuration to achieve strong backscattered signals for better damage measures, and a few reasonable configurations were suggested. Hence, it was found that sensing with sensor groups was possible in spite of coupling effects. However, the sensitivity of the developed sensor had to be tested for a wide range of cracks to fully validate the feasibility of the sensor for field implementation.
In this chapter, a newly developed method for crack propagation detection testing of future RFID-based sensor developments is explained. This method was designed in order to verify that this sensor is capable of detecting propagating cracks on metallic surfaces. This method compares the results of the backscatter power-based damage indicator to physical characteristics of progressive linear elastic fracture with high precision using digital image correlation. The physical characteristics measured to evaluate this sensor are vertical strain and crack opening (or crack width). A correlation between the damage index and crack opening has been found using data from the crack propagation tests for single- and multi-sensor configurations that were determined from the previous study. Considering the low cost of the developed RFID-based sensor, the crack detection performance is reasonable and shows strong potential for field implementation on metallic structures.

7.2 Theoretical background
The theoretical background of the developed sensing system has been summarized in this section for completeness.

7.2.1 Damage Sensitive Measure: Received Signal Strength Indicator
A basic setup for RFID inventory sessions consists of a reader/transmitter antenna connected to an RFID reader, in turn connected to a controlling computer. The RFID reader translates the signals sent by the controller to the reader/transmitter antenna to send and collect radio frequency signals to a target. For the specific RFID-based crack sensor developed, this target is the RFID tag. The principal indicator for damage in the RFID crack sensor developed is backscatter power. Backscatter power is dependent on the reader/transmitter antenna gain \( G \), the read/transmit distance \( R \), the transmitted power \( P_T \), the wavelength of the read/transmitted signal \( \lambda \), and the radar cross-section of the RFID tag \( \sigma \). This relationship is described by the radar range equation for a monostatic application:
\[ P_R = P_T \frac{G^2}{4\pi} \left( \frac{\lambda}{4\pi R} \right)^2 \sigma \]  

(7.1)

where \( P_R \) is the backscatter power. Resulting from previous studies [12], a standard read/transmit distance of 0.91 m (3 ft) has been chosen considering the range of wavelengths available in the RFID equipment to attain uniform and comparable results. All variables are maintained constant during testing except radar cross-section. Thus, backscatter power is directly dependent on this property of the RFID tag, which is affected by changes in the magnetic environment near or within the RFID tag.

Most RFID equipment reports backscatter power in a logarithmic expression known as received signal strength indicator (RSSI). The equipment used in this study reports RSSI in units of dBm as:

\[ RSSI = 10 \log_{10} \left( \frac{P_R}{\text{1mW}} \right) \]  

(7.2)

The following sections describe the parts of the sensor prototype, the backscatter power-based damage indices, and the usage of multiple sensors for increased monitored area.

### 7.2.2 Single-sensor prototype

The parts of the RFID-based crack sensor are graphically shown in Figure 7.1. The sensor is first assembled by adhering an Alien Technology ALN-9662 Short Inlay tag to a 12.7mm (0.5 in) thick ethyl-vinyl acetate (EVA) foam substrate. The substrate serves the purpose of increasing the radiation efficiency of the sensor, such that if backscatter power decreases greatly upon damage, it may still be measured by the reader antenna. This commercial tag is 70 mm long and uses a Higgs 3 EPC Class 1 Gen 2 RFID tag integrated circuit (IC). After several preliminary tests for crack propagation, it was found that the adhesive would have minimal interference with the changes in radar cross-section of the RFID tag if it was applied to the outermost portion of the tag and EVA foam, as shown by the darker shaded areas shown in Figure 7.1.
7.2.3 Damage indices

The quantitative indicators for damage are based on normalized changes in backscatter power. The total damage index is normalized by the initial reading in backscatter power while the temporal damage index is normalized by the previous backscatter power. In mathematical terms, the total damage index is:

\[ \Delta P_{\text{total}}(\%) = \left( \frac{P_i - P_{\text{baseline}}}{P_{\text{baseline}}} \right) \times 100 \]  

where \( P_{\text{baseline}} \) is the backscatter power in mW of the initial reading of backscatter power and \( P_i \) is the backscatter power in mW of the current state. The temporal damage index is:

\[ \Delta P_{\text{temporal}}(\%) = \left( \frac{P_i - P_{i-1}}{P_{i-1}} \right) \times 100 \]  

Previous tests of damage sensitivity [12] have shown that the algebraic sign of the total damage index indicates the location of crack formation with respect to the IC. These previous tests demonstrated that cracks occurring on the monitored surface to the left of the IC would increase backscatter power with respect to the original reading while crack occurring to the right of the IC would have the opposite effect. The crack propagating tests presented in this chapter demonstrate a similar trend when cracks occur on the monitored surface below or above the tag IC. These tests also demonstrate the relevance of the newly defined temporal index to early stages of crack formation.
7.2.4 Multiple sensor configurations

One of the main objectives of the crack sensor is to be capable of monitoring larger areas. In order to accomplish this, sensor groups must be used. However, sensors found in larger arrays can have detrimental consequences in the radiation efficiency of each sensor due to coupling or proximity effects [208]. Higher radiation efficiency is desired so that changes to structural integrity do not reduce backscatter power in the sensor to values less than the lower RSSI range of the reader. Previous studies performed on this RFID-based crack sensor showed that a 2x1-sensor configuration yields the greatest amount of backscatter power and that a 2x2-sensor array provided the best balance between radiation efficiency and pervasiveness [12]. These studies also determined the optimal spacing of sensors in arrays for increased radiation efficiency. All tests reported in the present paper use the optimally spaced 2x1- and 2x2-sensor configurations determined from the above-mentioned previous studies.

7.3 Description of crack propagation tests

Crack propagation tests were conducted to demonstrate the capability of the commercial RFID-based crack sensor to detect real crack propagation. Two sizes of specimens were designed to comply with a linear elastic, plane-stress fracture toughness test [210] so that a crack would initiate next to a sensor and propagate beneath it. Each specimen type was designed to accommodate single- or multiple-sensor configurations; the dimensions and tension force location of both specimen types A and B are shown in Figure 7.2. A space of 25.4 mm (1 in) by 50.8 mm (2 in) was allowed for the grips of the testing machine, indicated by the shaded regions with arrows. The inclusion of anti-buckling plates prevented out-of-plane deformation in the plate during testing. All plates and anti-buckling plates were 6.35 mm (0.25 in) thick. The materials of the plate and the anti-buckling plates were highly machinable MIC6® aluminum and 6061-T6 aluminum, respectively. A total of three samples were examined, each with a different sensor configuration: (1) Sample 1: a single sensor on a type A specimen, (2) Sample 2: a 2x1-sensor array on a type B specimen, and (3) Sample 3: a 2x2-sensor array on a type B specimen. The spacing of the sensors
in arrays is shown in Figure 7.3. Each sensor or sensor group was mounted to the specimen right at the end of the pre-crack line.

![Figure 7.2. Crack propagation specimens: (a) Type A, (b) Type B.](image)

![Figure 7.3. Multiple-sensor layouts in: (a) Sample 2, (b) Sample 3.](image)
Each crack sensor was assembled and mounted to the clean aluminum surface and allowed to cure to reach full bonding strength. The aluminum surfaces were sandblasted using a Cyclone FT3522 benchtop blast cabinet to an even and clean finish. The bonding process of sensor assembly and mounting was performed using Loctite Epoxy Quick Set, a two-part adhesive consisting of epoxy resin and a hardener that achieves a tensile shear strength of 23.68 ± 0.40 MPa (3437 ± 58 psi) in 24 hours. Epoxy resin was chosen for the assembly of the present crack sensor because it is typically used in strain-sensitive applications, such as crack sensing with optical fiber. Epoxy resin had also been manually tested for interference with RFID power signals in the laboratory and no significant effect had been found.

Figure 7.4 shows the testing setup for these experiments. A PC with Impinj MultiReader software was connected to an Impinj Speedway Revolution R420 UHF RFID Reader [211]. A high gain circular right hand polarized patch antenna was connected to the reader. The reader antenna was maintained level with the crack sensor at a read distance of 0.91m (3 ft). Tensile testing was completed with an ADMET eXpert 1655 Hydraulic Universal Test System (Norwood, MA) with a 250kN load cell. The grips of the instrument were clamped flush with the interior edge of the anti-buckling plates. Images to determine strain maps and crack width were collected with a Canon EOS 70D camera using a Neewer digital timer remote EZa. Strain mapping was performed on all samples using digital image correlation (DIC). This would allow the detection of strain concentrations at the crack tip to track damage occurrence during the test and aid in the correlation of changes in backscatter power. The samples were carefully sprayed with charcoal-colored, fine-textured spray paint on a white primer to obtain a speckle pattern on the side opposite to the crack sensors in the area of crack propagation, as shown in Figure 7.5. A red-filtered LED lamp was placed at 1 m (3.28 ft) from the specimen to render a clearer speckle pattern by providing greater contrast on the spray-painted side of the sample. The camera, lamp and reader antenna were all placed on tripods to be level with the area of interest.
The method used to propagate the crack was designed specifically to evaluate the performance of any RFID-based sensor in characterizing gradual crack damage. The tests were conducted at a constant grip displacement rate that allowed the test system software to record data up to a point close to complete fracture across the sample width. Sample 1 was displaced at a rate of 0.15 mm/min (0.006 in/min),
Sample 2 at a rate of 0.38 mm/min (0.015 in/min), and Sample 3 at a rate of 0.15 mm/min (0.006 in/min) up to a grip displacement of 1.27 mm (0.05 in) and 0.30 mm/min (0.012 in/min) afterwards. Systematic pauses at equidistant displacements were made to measure backscatter power for approximately 15 seconds. These pauses were made every 0.13 mm (0.005 in) in Sample 1, every 0.32 mm (0.0125 in) in Sample 2, every 0.13 mm (0.005 in) up to 1.3 mm (0.05 in) in grip displacement in Sample 3, and every 0.25 mm (0.01 in) after 1.3 mm (0.05 in) in grip displacement in Sample 3. High-resolution photographs of the speckle pattern for DIC were also collected during these pauses. An initial backscatter power reading before beginning the tests was taken as a baseline measurement. Samples 1 and 3 totaled 40 steps of increased grip displacement before reaching the maximum number of data points allowed by the controller for the hydraulic testing machine. Sample 2 totaled 22 steps of increased grip displacement after which the crack propagated completely across the specimen. Figure 7.6 shows all failed samples after testing. The dots on the sensors indicate the IC of the sensors under which the crack propagated. The temperature and relative humidity recorded at the times of testing are the following: 32.2ºC (90ºF) and 58% relative humidity for Sample 1, 31.7ºC (89ºF) and 61% relative humidity for Sample 2, and 26.7ºC (80ºF) and 51% relative humidity for Sample 3.

![Figure 7.6. Failed samples after crack propagation tests: (a) Sample 1, (b) Sample 2, (c) Sample 3.](image)
7.4 Results of crack propagation tests

The digital imagery of each step increasing the severity of fracture provided the means to determine the physical properties of crack propagation that would be correlated to changes in backscatter power. First, crack width (or crack opening) could be determined by performing precise measurements on these images. Crack opening was measured at the edge of the sensor that first encountered the path of the crack. The precision of these measurements was 0.0001 mm. Next, DIC analysis was performed to track the plastic zone formation in front of the crack throughout the test. The program used to perform this analysis was Ncorr v1.2, an open source 2D MATLAB add-on [212]. DIC uses image processing techniques to track small subsections of current images, called subsets, in relation to an undisturbed reference image. DIC parameters include subset radius and spacing, strain radius, and iterative solver options that include a difference norm and iteration number cutoff. The problem is solved using non-linear optimization with the Gauss-Newton non-linear iterative least squares method. The software calculates strain and displacement using Green-Lagrange and Euler-Almansi strain tensors and displacement gradients.

The crack propagation tests performed have provided sufficient information to determine the efficiency of the RFID-based sensor for crack detection and of the test method to evaluate such a sensor. First, the method of crack propagation testing provides ample information on the physical characteristics of the propagating crack to correlate the damage indicator of the sensor to the physical phenomenon of damage. The samples designed allow a simple approach to crack propagation monitoring executable in a laboratory setting for various sensor deployment sizes. The loading protocol allows the propagation of a crack in plane-stress condition as a static test without deforming the samples plastically anywhere but in the plastic zone in front of the crack. The collection of digital images provides for precise measurements of crack opening and strain mapping. This information can be also used to measure crack length and displacement mapping in the region if needed.
7.4.1 Analysis of single-sensor configuration

Figure 7.7 shows the vertical strain maps of Sample 1 in Steps 8 and 13 (corresponding to grip displacements of 1.016 mm (0.040 in) and 1.651 mm (0.065 in), respectively) according to the Green-Lagrange displacement gradient method. These maps have matching scales given by the 20 mm scale bar at the inferior left corner of each. The strain map before the plastic zone moves beneath the sensor at 1.016 mm (0.040 in) grip displacement at Step 8 (Figure 7.7a) reveals a plastic zone shape consistent with mode I loading. In order to approximate the failure in this test to the type of crack failure that typically occurs on high strength steel, it is necessary to achieve a linear elastic mode of failure. Linear elastic fracture is defined as a type of fracture that occurs when the size of the yielded material is confined to a small region [213]. The distance in front of a crack tip where yield stresses occur in small-scale yielding is defined by:

\[
r_p = \frac{1}{\pi} \left( \frac{K_{IC}}{\sigma_{YS}} \right)^2
\]

where \( K_{IC} \) is the fracture toughness of the material and \( \sigma_{YS} \) is the yield strength of the material. Since the fracture toughness for MIC6® aluminum is unknown, it was assumed that the ratio of fracture toughness to yield strength of MIC6® aluminum was the same as that of 6061-T6 aluminum. Analytically, this would mean that the plastic zone size should be no more than 2.6 mm. The plastic zone size in Sample 1 right before the crack propagated was 2.3463mm. Hence, the size of the plastic zone reveals that non-linear material deformation was confined to this small region, indicating linear elastic fracture. Figure 7.7b shows the plastic wake along the edges of the crack at 1.651 mm (0.065 in) grip displacement at Step 13. The plastic wake was characteristic of a crack that is propagating in steady state.
Figure 7.7. Strain maps of Sample 1 (color scale bar in mm/mm): (a) Step 8 before crack propagation, (b) Step 13 at steady state crack propagation.

The program used for DIC allowed to determine strain at each pixel of the analyzed region. Therefore, strain was measured at the edge of the sensor under which the crack first opened. Figure 7.8 shows the relationship between the maximum strain measured along the edge of the sensor and crack opening at the same edge of the sensor. The maximum strain of 4.34% coincided with the moment the crack tip opening was greater than zero (crack initiation) at the edge of the sensor. After this, the strain wake around the crack dropped as the crack first opened until it finally remained stable at about 2.7% as the crack widened, a result of steady state crack growth.

Figure 7.8. Strain vs. crack opening at the entering edge of the sensor of Sample 1.
In order to correlate the physical changes caused by the crack to the total damage index of the sensor, the change in backscatter power with respect to the undamaged state was plotted against crack opening in Figure 7.9. The circle-shaped data points represent the measured data during the test. The dashed line represents the average value of power change or crack opening and the solid lines represent the lower and upper bounds of one standard deviation from these average values. The relationship between change in backscatter power and crack opening appears to be piecewise. After power increased abruptly and remained approximately stable, the crack first opened to 0.0650 mm and propagated beneath the sensor. Backscatter power change then abruptly increased again, such that crack opening was nearly stable at that point. Then power change remained stable a second time while crack opening continued to increase and finalized with another abrupt increase in backscatter power.

Figure 7.9. Total damage index vs. crack opening at the entering edge of the sensor of Sample 1.

A Heaviside function has been determined based on this data that allows us to correlate the crack opening condition to the change in power for a single-sensor configuration. This function can be defined as follows:
\[ a_g (\Delta P) = \begin{cases} 
0, & \Delta P < \Delta P_1 \\
(0, \overline{a_{g1}}], & \Delta P = \Delta P_1 \\
\overline{a_{g1}}, & \Delta P_1 \leq \Delta P \leq \Delta P_2 \\
[a_{g1}, \overline{a_{g2}}], & \Delta P_2 \leq \Delta P \leq \Delta P_{\text{max}} 
\end{cases} \]  \tag{7.6}

where \( a_g \) is the crack opening, \( \Delta P \) is the change in power, \( \overline{a_g} \) is the average crack opening at the specified range of change in power, and \( \overline{\Delta P} \) is the average change in power at the specified crack opening range. Table 7.1 contains the values for the sample average change in backscatter power and crack opening for the single-sensor configuration in Sample 1.

**Table 7.1. Parameters for Heaviside Function of a Single-sensor Configuration (Sample 1).**

<table>
<thead>
<tr>
<th>Range</th>
<th>Average ( a_g ) (mm)</th>
<th>Standard deviation ( a_g ) (mm)</th>
<th>Average ( \Delta P ) (%)</th>
<th>Standard deviation ( \Delta P ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.8608</td>
<td>0.0759</td>
<td>5.68</td>
<td>1.98</td>
</tr>
<tr>
<td>2</td>
<td>1.4368</td>
<td>0.1341</td>
<td>23.1</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The interaction between the crack opening and the change in power can be explained as an interference between the tag antenna, which is a folded dipole structure with meandering element (a squiggle antenna), and the crack, which acts as a slot antenna. The crack presence acts as an additional radiating element that alters the electromagnetic field, thus changing the radiation efficiency of the system and returning a different backscatter power [214]. Since the geometry of a slot antenna on a ground plane with limited dimensions and the separation of elements in an array have a direct impact on the electric field, the changes in crack opening also have a direct impact on the radiation efficiency of the superposed RFID tag-based sensor. Therefore, changes in power can be associated to physical changes in crack opening.

In addition, the sensor did not only show to be sensitive to crack opening, but also to the early stages of crack initiation. Figure 7.10 through Figure 7.12 show the total damage index, temporal damage index, and strain at the edge of the sensor against the grip displacement, respectively. The first indication
of damage is shown by the dashed line in Figure 7.10 and in Figure 7.11. The abrupt change in power shown at 1.016 mm (0.040 in) grip displacement at Step 8 is very notable when the temporal damage index is used (Figure 7.11), marking early damage stages. The temporal damage index at this stage was 10.31%. Also, as shown in the strain map marking the location of the sensor in red dashed lines in Figure 7.13a, this instance corresponds to when the plastic zone was right at the edge of the sensor. At a grip displacement of 1.27 mm (0.05 in) at Step 10, the entire plastic zone is beneath the sensor and the crack is about to open at the edge of the sensor, as shown in Figure 7.13b. This second indication of damage is shown by the solid line in Figure 7.10 and in Figure 7.11. The information based solely on changes in backscatter power provided by the total and temporal damage indices makes way for an alternative method to characterize cracks when crack geometry is physically immeasurable or unavailable.

![Figure 7.10. Total damage index vs. grip displacement of Sample 1.](image1)

![Figure 7.11. Temporal damage index vs. grip displacement of Sample 1.](image2)
7.4.2 Analysis of multiple-sensor configurations

In order to understand the influence of having multiple sensors acting together to monitor a larger area, the same analyses for vertical strain and crack opening at the edge of the sensor where the crack entered first were performed on Samples 2 and 3. The trends in backscatter power change were also recorded for Samples 2 and 3 to determine their relationship to physical crack characteristics. The vertical strain maps shown in Figure 7.14a and Figure 7.15a show the plastic zone formation in front of the crack tip right before crack initiation in Sample 2 at a grip displacement of 0.9525 mm (0.0375 in) at Step 3 and in
Sample 3 at a grip displacement of 1.5240 mm (0.06 in) at Step 11, respectively. Figure 7.14b and Figure 7.15b show the plastic wake after the crack had propagated in steady state at 1.5875 mm (0.0625 in) in grip displacement at Step 5 in Sample 2 and at 2.0320 mm (0.08 in) at Step 13 in Sample 3. These maps have matching scales given by the 40 mm scale bar on the inferior left corner of each. The shape of the plastic zones in Figure 7.14a and Figure 7.15a are consistent with mode I loading. The maximum distance in front of the crack tip at which the fracture would conform to small scale yielding is 2.6 mm, as determined with Equation (7.5). The plastic zone size of Sample 2 was 2.0594 mm and the plastic zone size of Sample 3 was 2.6138 mm. Therefore, their size also conformed to the limitations stipulated by linear elastic fracture mechanics. The plastic wakes seen in Figure 7.14b and Figure 7.15b are characteristic of a crack that is propagating in steady state.

Figure 7.14. Strain maps of Sample 2 (color scale bar in mm/mm): (a) Step 3 before crack propagation, (b) Step 5 at steady state crack propagation.
Figure 7.15. Strain maps of Sample 3 (color scale bar in mm/mm): (a) Step 11 before crack propagation, (b) Step 13 at steady state crack propagation.

Similar to the single-sensor configuration, these larger samples presented maximum strain values at the edge of the sensor under which the crack first entered right before crack initiation, as shown by Figure 7.16 and Figure 7.17. The maximum strain recorded on Sample 2 was 2.16% while the maximum strain recorded on Sample 3 was 16.57%. The differences in magnitude of the strain can be due to the discontinuity in measurement between one frame and the next. Since strain was discretely measured at every 0.32 mm (0.0125 in) in grip displacement for Sample 2, it is possible that the true maximum strain value was closer to that of Samples 1 and 3, which had more closely spaced measurements every 0.13 mm (0.005 in) in grip displacement. Similarly, since measurements were taken discretely, the true maximum strain in Sample 3, which always occurs right before crack initiation, very likely occurred right before the recorded maximum strain value. The strain value of 16.57% in Figure 7.17 represents the strain immediately after crack initiation, before the strain wake is fully established. After the point of maximum strain, the crack opened and the strain wake around the crack remained constant at 0.365% in Sample 2 and at 0.1692% in Sample 3 as the crack steadily propagated in both samples.
It was found upon examination for the changes in backscatter power in all sensors that the sensor in a multiple-sensor configuration that consistently provided a trend in power change closest to that of the single-sensor configuration (Sample 1) was the one farthest from the area where the crack first started. In other words, the correlation between the physical damage and the change in power of Sensor A in Sample 2 and Sensor A2 in Sample 3 proved to be best for crack propagation monitoring. This phenomenon can be explained by the disruption in symmetry that the added crack has in the RFID sensor group. Asymmetric changes to the electromagnetic environment in a sensor group cause asymmetric changes to
the radiation efficiencies of the sensors. Since the crack acts as a slot antenna and changes the radiation
efficiency of each sensor, its closer proximity to Sensor B in Sample 2 and to Sensors B1 and B2 in
Sample 3 distorted the radiation patterns of these sensors greatly. Therefore, all results presented below
reference backscatter power changes to Sensor A in Sample 2 and Sensor A2 in Sample 3 only.

The relationships between crack opening and the total damage index for Samples 2 and 3 are
shown in Figure 7.18 and Figure 7.19. The circle-shaped data points represent the data collected during
the test. The dashed line represents the average change in power or crack opening while the solid lines
represent the upper and lower bounds of one standard deviation from the average values. It should be
noted that the inverse trend in power change between Sample 1 and Samples 2 and 3 is due to the location
of the crack with respect to the IC. In previous tests shown in Chapter 6 [12], it was found that the
location of a crack with respect to the RFID tag IC would have an effect in the direction of backscatter
power change, whether an increase or a decrease. Referencing to Figure 7.6, the black dot in each photo
indicates the location of the tag IC. Since the crack propagated above the tag IC in Sample 1, but beneath
the tag IC in Samples 2 and 3, the trends in power change are algebraically opposite.

![Figure 7.18. Total damage index at Sensor A vs. crack opening of Sample 2.](image-url)
The relationship between the total damage index and crack opening in both Samples 2 and 3 also appear to be piecewise and once again describable with a Heaviside function. In both cases, when power reached approximately a stable value, the crack began to open under the first sensor in the path of fracture. Sample 2 presents an abrupt change in total damage index before said stabilization. This first detected crack opening was 0.1733 mm in the 2x1-sensor array and 0.0922 mm in the 2x2-sensor array. In Sample 2 (Figure 7.18), there are 3 ranges in which power stabilized and later dropped suddenly with an approximately constant crack opening. Sample 3 presents 4 ranges where power stabilized in Figure 7.19. This 2x2-sensor configuration did not clearly show that at the end of these ranges there was a constant crack width at which power changed drastically, but rather each range consisted of a constant change in power.

A Heaviside function similar to that of a single-sensor configuration has been derived for the 2x1-sensor configuration as follows:

$$a_g(\Delta P) = \begin{cases} 
0, & \Delta P < \Delta P_j \\
\left[0, \bar{a}_{g1}\right], & \Delta P = \Delta P_j \\
\bar{a}_{g1}, & \Delta P_j \leq \Delta P \leq \Delta P_2 \\
\left[\bar{a}_{g1}, \bar{a}_{g2}\right], & \Delta P = \Delta P_2 \\
\bar{a}_{g2}, & \Delta P_2 \leq \Delta P \leq \Delta P_3 \\
\left[\bar{a}_{g2}, \infty\right), & \Delta P = \Delta P_3 \end{cases}$$  \quad (7.7)
All variables are as defined in Equation (7.6) and the values of the variable found experimentally are listed in Table 7.2. The parameters of the Heaviside function are different for each sample because each sensor configuration has a different interference and wave propagation pattern.

Table 7.2. Parameters for Heaviside Function of a 2x1-sensor Configuration (Sample 2).

<table>
<thead>
<tr>
<th>Range</th>
<th>Average ( a_g ) (mm)</th>
<th>Standard deviation ( a_g ) (mm)</th>
<th>Average ( \Delta P ) (%)</th>
<th>Standard deviation ( \Delta P ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.6105</td>
<td>0.1553</td>
<td>5.60</td>
<td>0.993</td>
</tr>
<tr>
<td>2</td>
<td>1.7501</td>
<td>0.1920</td>
<td>-1.15</td>
<td>0.746</td>
</tr>
<tr>
<td>3</td>
<td>2.4895</td>
<td>0.1955</td>
<td>-5.55</td>
<td>0.264</td>
</tr>
</tbody>
</table>

Similarly, the Heaviside function for the 2x2-sensor configuration is the following:

\[
a_g(\Delta P) = \begin{cases} 
0, & \Delta P < \Delta P_1 \\
\left[0, a_{g1}\right], & \Delta P = \Delta P_1 \\
\left[a_{g1}, a_{g2}\right], & \Delta P = \Delta P_2 \\
\left[a_{g2}, a_{g3}\right], & \Delta P = \Delta P_3 \\
\left[a_{g3}, a_{g4}\right], & \Delta P = \Delta P_4 
\end{cases}
\]  

(7.8)

Table 7.3 shows the values for the Heaviside function of the 2x2-sensor configuration.

Table 7.3. Parameters for Heaviside Function of a 2x2-sensor Configuration (Sample 3).

<table>
<thead>
<tr>
<th>Range</th>
<th>( a_g ) (mm)</th>
<th>Average ( \Delta P ) (%)</th>
<th>Standard deviation ( \Delta P ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.9351</td>
<td>9.30</td>
<td>11.8</td>
</tr>
<tr>
<td>2</td>
<td>1.3925</td>
<td>-8.37</td>
<td>1.58</td>
</tr>
<tr>
<td>3</td>
<td>2.2881</td>
<td>-23.8</td>
<td>8.02</td>
</tr>
<tr>
<td>4</td>
<td>3.1789</td>
<td>-39.3</td>
<td>5.18</td>
</tr>
</tbody>
</table>
A different Heaviside function has been determined for each configuration to estimate changes in crack opening based on changes in received power. Since low-cost RFID tags present large variability in backscatter power, the parameters of the Heaviside function can also vary per test. Therefore, rather than conducting a large number of tests to determine a unique range of values for these parameters, it is recommended to take the abrupt change in received power as an indication of crack damage, as provided by the temporal damage index.

Figure 7.20 through Figure 7.22 show the total damage index of Sensor A, temporal damage index of Sensor A, and strain at the edge of Sensor B in Sample 2 against the grip displacement, respectively. At a grip displacement of 1.5875 mm (0.0625 in) at Step 5, the temporal damage index achieves its maximum value of 4.148%, marking the abrupt change in power that indicates crack initiation. This stage is marked by a solid red line in Figure 7.20 and in Figure 7.21. This abrupt change in power occurs after the crack has moved completely under the sensor, as shown by the strain map with the marked location of Sensor B at a grip displacement of 1.5875 mm (0.0625 in) at Step 5 in Figure 7.23b. In the previous reading at a grip displacement of 1.27 mm (0.05 in) at Step 4, the crack has not yet opened at the edge of the sensor, but the entire plastic zone is beneath the Sensor B (Figure 7.23a). This stage is marked by a dashed line in Figure 7.20 and in Figure 7.21. This is a slight difference to the case of Sample 1, where the abrupt change in power occurred when the plastic zone was at the edge of the sensor. Such distortions in backscatter power are to be expected when more RFID tag antennas are used due to coupling effects. This result indicates the tradeoff between using a larger number of sensors to monitor a larger area and using a single sensor to achieve greater precision in the detection of crack initiation.
Figure 7.20. Total damage index vs. grip displacement of Sample 2.

Figure 7.21. Temporal damage index vs. grip displacement of Sample 2.

Figure 7.22. Strain vs. grip displacement at the entering edge of Sensor B of Sample 2.
Although the relationship between crack opening and damage index is clearly established for Sample 3, this 2x2-sensor array presented the most difficult case to detect the instance of crack initiation. Figure 7.24 through Figure 7.26 show the total damage index of Sensor A2, temporal damage index of Sensor A2, and strain at the edge of Sensor B1 in Sample 3 against the grip displacement, respectively. Sensor A2 shows the first salient temporal damage index of 24.11% at a grip displacement of 0.3810 mm (0.015 in) at Step 3, although the plastic zone has not yet formed at the edge of Sensor B1. The second instance occurs at a grip displacement of 0.7620 mm (0.030 in) at Step 6 with a temporal damage index of 16.80%. This is the instance in which the plastic zone is right at the edge of Sensor B1 (Figure 7.27a), showing the first indication of damage onset. This stage is marked by the dashed red line in Figure 7.24 and in Figure 7.25. The third time the temporal damage index peaks (17.15%) is at a grip displacement of 1.27 mm (0.050 in) at Step 10. At this stage, the plastic zone is entirely under Sensor B1, although the crack has not opened at its edge, as seen in Figure 7.27b. This stage of damage is marked by the dotted red line in Figure 7.24 and in Figure 7.25. The fourth time the temporal damage index peaks (22.80%) is at a grip displacement of 1.7780 mm (0.070 in) at Step 12. At this instance, the crack has opened beneath Sensor B1 (Figure 7.27c) and the total damage index begins to stabilize. This stage of damage is marked by the solid red line in Figure 7.24 and in Figure 7.25. The 2x2-sensor array was capable of indicating the
onset of damage when the plastic zone was at the edge of Sensor B1 at Step 6 and when the crack opened at the same edge at Step 12. However, the false indication of damage at Step 3 where the temporal damage index reported a relevant value was due to the distortions present in sensor groups, which become more relevant as the number of sensors increases. Therefore, the RFID-based crack sensor is more sensitive to crack formation when used as a single sensor, although groups can be used to detect crack formation with less precision without compromising sensitivity to crack opening.

Figure 7.24. Total damage index vs. grip displacement of Sample 3.

Figure 7.25. Temporal damage index vs. grip displacement of Sample 3.
Figure 7.26. Strain vs. grip displacement at the entering edge of Sensor B1 of Sample 3.

Figure 7.27. Strain maps of Sample 3 showing sensor location: (a) Step 6 with plastic zone at the edge of Sensor B1, (b) Step 10 with the plastic zone beneath Sensor B1, (c) Step 12 with the crack opened under Sensor B1.
Thus far, the crack propagation experiments performed on these low-cost RFID-based sensors have demonstrated their sensitivity to variations in crack opening and to early stages of crack formation. The temporal damage index is a tool that allows the detection of crack onset when salient values are attained and the total damage index is correlated to crack opening in a piecewise manner. Therefore, the performance of the developed sensor has been successfully validated using crack propagation tests.

In the future, further refinement of these damage indices are desired to systemize their usage for practical applications. In particular, threshold values to determine damage locations and quantities for the temporal index would need to be established for different steel element geometries and sensor configurations. In addition, further experiments to correlate the damage indices and other failure modes such as fatigue, shear, or rupture could be considered. Nonetheless, this study prepared a suggested experimental framework to validate the performance of RFID-based sensors.

7.5 Conclusions
The study presented in this chapter has shown the potential for field application of a passive UHF RFID-based crack sensor for crack propagation monitoring in metallic structures. The developed crack detection sensor is the first low-cost UHF RFID-based crack sensor that can detect cracks propagating in a metallic medium using wireless communication on passive sensors and simple power-based damage indicators. The crack sensor has been refined to be sensitive to crack propagation on metallic structures by redesigning the method of assembly and mounting and defining a new temporal damage index. A new method to evaluate the sensitivity to crack propagation of an RFID-based crack sensor using backscatter power as the principal measurand was designed. This method uses ASTM standards for specimen dimension and a rigorous test execution method to measure the physical properties of the propagating crack with high precision. This method provided information to determine that the type of fracture analyzed in these tests was linear elastic, which is the type of local failure to be expected in areas of high stress concentration in steel members. The crack sensor was tested to correlate the defined backscatter
power-based total damage index to the crack opening on the specimen. It was found that the total damage index achieves a stable condition when the crack opens at the edge of the sensor first encountering the propagating crack. Another index, the temporal damage index, was also developed, and marks the instant in which said crack damage is initiating. The sensor can also be used in an array to increase monitoring pervasiveness without compromising the quality of sensing. However, the damage indicators of sensors in an array can be distorted when a crack forms, so antennas of sensors in an array farther from the damage zone showed to be most effective to this purpose, thus showing that multiple-sensor arrays are useful for indirect crack detection, while single-sensor configurations are feasible for direct crack detection and better sensitivity. A single sensor was capable of detecting a crack opening as narrow as 0.0650 mm. The 2x1-sensor array detected a crack 0.1733 mm wide and the 2x2-sensor array detected a crack 0.0922 mm wide. Therefore, these findings provided a new viable, low-cost, wireless passive crack propagation sensor on metallic structures, showing a great potential for crack detections of steel bridges in the field.
Chapter 8 Conclusions and Future Work

This dissertation has presented a great deal of work to advance SHM and NDE systems for critical load-carrying members in bridges. The idea of a CPS for civil structures was revisited and its components more clearly defined. To expound in the specifics of an SCPS, several projects were undertaken to provide valuable SHM information on critical load-carrying members that can aid in structural control decisions. Using vibration-based measurements, the ED damage identification method was experimentally validated and was modified to increase accuracy and automation to reduce false positive indications of damage while providing excellent results in determining existence, location, and quantification of damage near simulated cable anchor zones and in an laboratory-scale 5-story shear building. These findings and developments represent a step towards the implementation of the ED method for damage identification in real cable structures.

As future work, the modified ED method can be further validated on a real cable structure to verify findings from the simulation. Since a wireless cable tension estimating system has been successfully developed and validated via a laboratory-scale experiment and field data from a truss arch suspension bridge, this method could be incorporated into the smart wireless system microcomputer and use the same vibration data to perform damage identification as that used to estimate cable tension. The experiments performed for cable tension estimation allowed to further verify the effectiveness of the onboard algorithms and to compare the tension estimation methods. It was found that Methods 1 and 2 are the most accurate to perform tension estimation because they consider bending stiffness and use empirically determined coefficients in their formulations. Method 3 is conservative when using the fundamental frequency, but becomes more accurate with higher-mode frequencies in the examined cables. Similarly, taut string theory offers conservative tension estimations. However, when longer cables are used, estimations calculated with taut string theory or Method 3 become more accurate. This is so because the assumption of an ideal taut string becomes true in longer cables and the role of bending stiffness
becomes less relevant. The validation performed using field-data also allowed the users to identify suspension cables that were excessively loaded, demonstrating the relevance of performing regular cable tension load condition assessment. Performing similar assessments on the automated ED method and even comparing the method to other rapid vibration-based damage identification methods would be beneficial to expand the SHM capabilities of the developed smart wireless system.

Using a single-board microcomputer and peripheral sensors, the wireless tension estimating system effectively collects acceleration time history from a field-ready USB accelerometer, calculates PSD, generates ready-to-view PSD and acceleration time history plots, and selects natural frequencies from the PSD plot automatically using the peak-picking method. It also allows user inputs for accelerometer settings, PSD parameters, cable properties, and final natural frequency selection for quality assurance. The system also calculates a linear model for natural frequency fitting using the final selected natural frequencies and calculates cable tension using four different vibration-based tension estimating methods. All these actions can be performed remotely, a great convenience for field usage, since it can be operated from a remote computer via a WLAN. This wireless system can further be expanded to incorporate additional monitoring features, such as vehicle counting, a weather station, and further SHM vibration-based methods, such as the automatic ED damage identification method. Additional work with the smart wireless system can include interfacing the microcomputer with additional control and sensor peripherals to monitor other features, such as scour in bridge piles, corrosion, expansion joint displacements, and more. The development of this system poses a relevant advancement in the practical usage of low-cost and simple multi-sensory technology for reliable SHM that can be useful to promote structural control via SCPS technologies.

In addition to vibration-based SHM, a crack sensor was developed to provide a simple, low-cost, wireless, and sensitive means to detecting cracks on metallic surfaces. Since crack damage on steel bridges is often difficult to detect, an RFID-based crack sensor was successfully developed using low-cost commercial tags and its performance was validated with comprehensive laboratory-scale experiments. The developed crack detection sensor is the first low-cost UHF RFID-based crack sensor that can detect
cracks propagating in a metallic medium using wireless communication on passive sensors and simple power-based damage indicators. The method of assembly and mounting was defined and total and temporal damage indices were established to detect initial crack stages and to characterize crack width as a crack grows. A new method to evaluate the sensitivity to crack propagation of an RFID-based crack sensor using backscatter power as the principal measurand was also designed. This method uses ASTM standards for specimen dimension and a rigorous test execution method to measure the physical properties of the propagating crack with high precision. This method provided information to determine that the type of fracture analyzed in these tests was linear elastic, which is the type of local failure to be expected in areas of high stress concentration in steel members. The crack sensor was tested to correlate the defined backscatter power-based total damage index to the crack opening on the specimen. It was found that the total damage index achieves a stable condition when the crack opens at the edge of the sensor first encountering the propagating crack. Another index, the temporal damage index, was also developed, and marks the instant in which said crack damage is initiating. The sensor can also be used in an array to increase monitoring pervasiveness without compromising the quality of sensing. However, the damage indicators of sensors in an array can be distorted when a crack forms, so antennas of sensors in an array farther from the damage zone showed to be most effective to this purpose, thus showing that multiple-sensor arrays are useful for indirect crack detection, while single-sensor configurations are feasible for direct crack detection and better sensitivity. A single sensor was capable of detecting a crack opening as narrow as 0.0650 mm. The 2x1-sensor array detected a crack 0.1733 mm wide and the 2x2-sensor array detected a crack 0.0922 mm wide. Therefore, these findings provided a new viable, low-cost, wireless passive crack propagation sensor on metallic structures, showing a great potential for crack detection of steel bridges in the field.

Yet, further work is necessary to bring this crack sensor to a practice-ready state. The sensor performance needs to be evaluated under environmental uncertainties for field deployment. These tests require preparation in various areas that in and of themselves can pose additional research projects as part of the development process of the crack sensor. First, sensor mounting in the field may have to be
revisited for field application. One of the biggest challenges found in RFID-based sensors deployed in the field is mounting to the toes of welded connections. A way to mount the sensor such that an uneven toe weld does not impede functionality for damage detection is necessary. Second, weld defects can sometimes present multiple cracks instead of the usual single, clean crack line that is done in laboratory tests. The effect of multiple cracks on backscatter power changes needs to be assessed. Third, the effect of surface cracks that do not open the monitored material through the entire depth of the member needs to be examined, since all cracks so far induced have been full depth. Fourth, the effect of live load during in-service monitoring needs to be further assessed. From laboratory experience, it is known that live load will open cracks in tension regions further, and thus it should have a direct effect in backscatter power. It is possible that the inclusion of live load during monitoring may even be beneficial for crack detection, but the verification of this hypothesis is necessary. Fifth, the mechanical influence of the substrate EVA foam material must be explored. If confronted with environments that could affect the size of this very elastic layer (for instance, a drastic temperature gradient), it is possible that the induced strain on the RFID tag may affect backscatter power. Sixth, the sensitivity of the sensor to cracks increasing in length over time without changing crack width needs to be assessed, since cracks with these geometric characteristics are very common in fatigue-prone regions. The relationship between crack length and backscatter power could be established while maintaining the variable of crack width constant. Seventh, environmental factors such as humidity, temperature, and vehicle presence (reflective surfaces) may interfere with the radiation efficiency of the RFID tags, so this must be quantified and accounted for. Eighth, since crack-prone areas in bridge girders are usually difficult to access, new systems that can automatically register backscatter power signals from the RFID tags should be designed. These can range from a robot with an embedded reader antenna rolling along girder flanges and measuring backscatter power at predetermined stations, to vehicles with a mounted reader antenna that can cross under the bridge and perform a similar task. Finally, the mathematical model correlating backscatter power change with crack width must be statistically explained using probabilities, so more samples using the same
sensor configurations must be repeated so the model can become valid for a larger sample size and thus be representative of the population of each configuration.
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Appendix A Raspberry Pi SHM Station Manual

Before you begin:
The instructions below are meant to be for the Raspberry Pi 3 Model B SHM station from the Smart Infrastructure Laboratory of the University of Connecticut. The IP address and password of said station is for the exclusive use of the members of the Smart Infrastructure Laboratory and can only be used for University-related research. Instances in which {username}, {IP address} and {password} are indicated refer to these private credentials. Please contact Dr. Shinae Jang at shinae.jang@uconn.edu to request credentials to connect to the SHM station.

A-1 Connection
1. Plug the Raspberry Pi (RPI) to a power source. The RPI should load automatically. The RPI should automatically connect to the UCONN-GUEST network.
2. Establish a secure shell (SSH) connection.
   a. Mac OSX10.7 Yosemite
      i. Open a Terminal window (different window to SFTP connection).
      ii. Type
          ssh {username}@{IP address}
      iii. Type the password
      iv. The following should now display:
          {username}@raspberrypi:~$
   b. Windows 7
      i. Run putty.exe
      ii. Type the IP address in the Host Name field.
      iii. Type the user name.
      iv. Type the password.
      v. The following should now display:
         {username}@raspberrypi:~$
3. Establish a secure file transfer protocol (SFTP) connection.
   a. Mac OSX10.7 Yosemite
      i. Open a Terminal window (different window to SSH connection).
      ii. Type:
          sftp {username}@{IP address}
      iii. Type the password.
      iv. To close the SFTP connection, type:
          bye
      v. Alternatively, use FileZilla following the steps for Windows 7.
   b. Windows 7
      i. Open FileZilla.
      ii. Type the following to the following fields:
          Host: sftp://{IP address}
          Username: {username}
          Password: {password}
          Port: 22
      iii. Click “Quick Connect”
4. Update the current operating system.
   sudo apt-get update
5. To power off the RPI, type the following to the SSH connection window:
   sudo poweroff
   sudo shutdown now
A-2 Acquiring acceleration data
1. Connect the GCDC USB Accelerometer, Model X2-2 to any USB port. Use the extension cable if needed.
2. Through the SSH connection, change the directory to that of the accelerometer’s memory:
```bash
cd /media/pi/X02-C2D8
```
3. Open the configuration file for the accelerometer:
```bash
nano config.txt
```
4. Change the properties of data collection according to the features of the monitored structure. For a full explanation of the configuration parameters, refer to Section 3.2.4 System Configuration Options, in the document “GCDC_X2-2_User_Manual.pdf”. The most relevant features are:
   a. samplerate – sets the sampling rate to be used during sensing
   b. gain – can be set to “low” or “high”. The low sensitivity mode sets the range to ±2g while the high sensitivity mode sets it to ±1.25g.
5. Close the configuration file by typing Ctrl+X. To save the changes, type “Y” and hit “Enter”.
6. Change the directory back to the home folder:
```bash
cd /home/pi
```
7. Begin acceleration collection by typing the following:
```bash
sudo ./gcdcTool --raw={numpts}>{filename}.csv
```
This command will execute the human interface device (HID) application “gcdcTool”, which will start the accelerometer immediately. It will collect the given number of data points ({numpts}) of raw acceleration data and save them into a .csv file entitled {filename}.

A-3 Transferring files
1. You may only transfer files between the SHM station (remote site) and the local computer after performing an SFTP connection (see Section A-1 Connection).
2. FileZilla:
   a. Choose a destination folder on the local computer (local site) on the left panel.
   b. Navigate to the path “/home/pi” on the SHM station (remote site) on the right panel.
   c. Drag the desired files from the remote site and drop at the local site.
   d. To place a file on the SHM station, drag the desired files from the local site and drop at the remote site.
3. Terminal window on Mac OS X:
   a. Choose a destination folder on the local computer:
      ```bash
      lcd {filepath}
      ```
   b. Choose the path from which the files will be transferred:
      ```bash
      cd /home/pi
      ```
   c. Save a file on the local computer. For example:
      ```bash
      get {filename}.csv
      ```
   d. To place a file on the SHM station:
      ```bash
      put {filename}.csv
      ```

A-4 Calculating cable tension
1. Collect the acceleration signal of the cable (see Section A-2).
2. Through an SSH connection window, make sure the current directory is set to the location of the file “inputs.m”:
```bash
cd /home/pi
```
3. Open the Wolfram package “inputs.m” to edit:
```bash
nano inputs.m
```
4. Enter the input variables:
   a. Direct – file path to CSV acceleration file
b. Filename – file name of the CSV acceleration file
c. Overlap – percent of window overlap for PSD
d. blur – Standard deviation of Gaussian blur for finding peaks in PSD plot
e. blurdb – Standard deviation of Gaussian blur for finding peaks in PSD plot (in dB scale)
f. nfft – number of data points for fast Fourier transform
g. fs – sampling frequency in CSV acceleration file
h. corr – correction factor for accelerometer gain (13108 for high gain; 6554 for low gain)
i. BC – boundary condition of cable ends; 1 = hinged-hinged, 2 = fixed-fixed, 3 = hinged-fixed
j. l – cable length (ft)
k. w – linear weight of cable (lb/ft)
l. Ela – Young’s modulus of cable (ksi)
m. Ine – second moment area of cable cross-section (in^4)
n. A – cable cross-sectional area (in^2)
o. θ – angle of cable inclination with respect to the horizon (deg)
p. To – expected cable tension (kip)
q. s – cable sag (in)

5. Once all values have been entered, hit the enter/return button and save changes
6. Open a second SSH connection window and type
   `wolfram`
   to open the Wolfram Kernel. Once set, the following will show:
   ```wolfram
   In[1]:
   ```
7. To run the inputs package, type:
   ```wolfram
   Import["inputs.m"]
   ```
8. To calculate the PSD plot, type:
   ```wolfram
   Import["PSD.m"]
   ```
   This will generate the following files that can be transferred via SFTP (see section A-3):
   a. AccelHist.jpeg – JPEG file of the acceleration time history
   b. PSD.jpeg – JPEG file of the PSD plot with automatically selected peaks in red
   c. PSDdb.jpeg – JPEG file of the PSD plot with automatically selected peaks in red (in dB scale)
   d. timehistory.txt – Text file of the acceleration time history data
   e. output.txt – Text file with the values of the selected natural frequencies (from simple and dB scale PSD plots)
9. After verifying the PSD plots and selected natural frequencies, if the plot or natural frequency selection is unsatisfactory, return to steps 3-5 and modify the variables blur, blurdb, Overlap, and nfft as needed. Run steps 7-8 until the plot and selection reach the most satisfactory result.
10. In the first SSH connection window, type:
    ```bash
    nano Freq.m
    ```
    and enter the final selected natural frequencies to be used for linear regression.
11. Hit the Enter/Return button and save the file Freq.m after editing.
12. In the second SSH connection window, type:
    ```bash
    Import["Freq.m"]
    Import["linreg.m"]
    The fitted natural frequencies should be displayed.
    Import["tautstring.m"]
    Cable tension according to taut string theory should be displayed.
    Import["cabletensionzui.m"]
    Cable tension according to Zui et al. (1996) (Method 1) should be displayed.
    Import["cabletensionren.m"]
    Cable tension according to Ren et al. (2005) (Method 2) should be displayed.
    Import["cabletensionhuang.m"]

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Cable tension according to Huang et al. (2015) (Method 3) should be displayed.

A-5 Troubleshooting:
1. The RPi is not loading when I connect it to a power source. No connections are possible (SSH, SFTP or VNC).
   Option #1:
   Disconnect the power. Wait about 15 seconds. Reconnect. Check if it loads.
   Option #2:
   Connect the RPi to peripherals (monitor through HDMI, keyboard and mouse). After powering the RPi, verify boot up screen. If there are many markings in red, this may mean that the data card has been corrupted. To restore the card image, refer to Section A-6.
   Option #3:
   Connect the RPi to peripherals (monitor through HDMI, keyboard and mouse). If the GUI loads, verify that the RPi is connected to the network (UCONN-GUEST). Do this by clicking on the Wi-Fi symbol to view the list of networks.

2. The VNC connection is refused, but SSH and SFTP are working.
   Through the SSH connection, update the software:
   ```
sudo apt-get update
```
   Install the latest version of VNC Connect:
   ```
sudo apt-get install real-vnc-server real-vnc-viewer
```
   Enable the VNC connection:
   ```
sudo raspi-config
```
   Select “Advanced Options”, then “VNC”, select “Yes” to the question “Would you like the VNC Server to be enabled?”. The VNC Server should now be enabled.

For more information refer to: [https://www.raspberrypi.org/documentation/remote-access/vnc/](https://www.raspberrypi.org/documentation/remote-access/vnc/)

A-5 Raspberry Pi image backups

A-5-1 Back up current image from SD card (Mac OS X)
1. Verify the disk name of the SD card under Disk Utility. Disk is usually under the name disk2s2, but can be referred to as disk2. This can also be done by typing the following into a Terminal window:
   ```
df -h
```
2. Open a Terminal window.
3. Type:
   ```
sudo dd bs=4m if=/dev/rdiskname of=/directory_address/imagename.img
```
   Example:
   ```
sudo dd bs=4m if=/dev/rdisk2 of=/Users/rosanamartinez-castro/Desktop/raspbian_05_04_2016.img
```
4. Information on transfer status may be seen by typing: Ctrl+T

A-5-2 Write image onto SD card (Mac OS X):
1. Verify the disk name of the SD card under Disk Utility. Disk is usually under the name disk2s2, but can be referred to as disk2.
2. Open a Terminal window.
3. Type:
   ```
diskutil unmountDisk /dev/disk2
sudo dd bs=1m if=/directory_address/imagename.img of=/dev/diskname
```
   Example:
sudo dd bs=1m if/Users/rosanamartinez-castro/Desktop/raspbian_05_04_2016.img of=/dev/disk2

4. Information on transfer status may be seen by typing: Ctrl+T
5. When transfer has been completed, eject disk by typing:
   sudo diskutil eject /dev/disk2