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STRUCTURAL HEALTH MONITORING AND SEISMIC IMPACT ASSESSMENT

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ABSTRACT

Civil infrastructure systems deteriorate generally in an uncontrollable speed. Inadequate operation, physical aging, and natural as well as manmade hazards threaten the safety and functionality of these systems. To assess the short-term impact due to hazards (e.g., earthquakes) and the long-term deterioration process due to physical aging and routine operation, continuous condition assessment and performance-based maintenance of civil infrastructure systems are necessary. Structural health monitoring (SHM) has been introduced into the regime of civil engineering as a potential and effective methodology for such an assessment. The fundamental principle of the method is that the existence of damages results in the changes in structural properties, such as mass, damping, and stiffness. These changes will alter both static and dynamic behaviors of structures and thus can be detected by measurements through distributed sensors. Deploying sensing devices (sensors, transducers) onto civil infrastructure systems are driven by rational engineering motivations, such as the potential merit in assuring safety and in saving future cost of maintenance. In this paper, components of SHM systems are reviewed and relationship between modal parameters and structural properties are developed. Relation of structural monitoring to seismic impact assessment is established and an ongoing instrumentation project, as an example, is described. Role of SHM in seismic impact assessment through an integrated, conceptual impact simulator is also discussed.
Keywords: Structural health monitoring, damage detection, seismic impact.

INTRODUCTION

Civil infrastructure is the artery of social and economic activities and an essential element of human well being. The investment on civil infrastructure systems (such as buildings, bridges, dams, reservoirs, tunnels, pipelines, airports, stadiums) usually represents the progress of civilization and their quality reflects the advances in engineering science and technology. Unfortunately, the degradation and deterioration process occurring of civil infrastructure is physically inevitable. Inadequate operation, physical aging, and natural as well as manmade hazards threaten the safety and functionality of these systems.

Civil infrastructure systems deteriorate generally in an uncontrollable speed. Even though these systems are designed to operate for long periods of time, unpredictable and/or uncontrollable factors reduce their expected performance and life cycle. Lack of performance of these systems may significantly impact a nation's economy. For example, the US Federal Reserve Board has concluded that the failure of civil infrastructure systems to perform at their expected level may reduce the national gross domestic product (GDP) by as much as 1% (FHWA, 1993). The loss of functionality of these systems can bring enormous and long-term impact to a nation and the recovery cost is huge. In order to avoid catastrophic results and reduce the loss, assessing the health condition of civil infrastructure for rational management actions has become a challenging hazard mitigation strategy for the engineering community. To observe the short-term impact due to special events (e.g., earthquakes) and the long-term deterioration process due to physical aging and routine operation, continuous condition assessment and performance-based maintenance of civil infrastructure systems are necessary.

Natural hazards, especially earthquakes, usually damage civil infrastructure in an unpredictable fashion. The induced, immediate property and human losses and the following economic loss would result in a disastrous impact. Traditionally, civil infrastructure systems are protected by seismic-resistant design from the threat of earthquakes. They are designed to possess sufficient strength and efficient energy-dissipation mechanism (ductility) in order to experience the vibration process safely. Whether a structure may fail or sustain in the event of an earthquake depends on the design, construction quality, local geology, and magnitude of the earthquake. For sustained-but-damaged structures (Fig. 1), knowledge of the structural response is crucial for evaluating damage severity (due to a specific earthquake event) and the performance of seismic-resistant design. Rehabilitation and retrofitting efforts also require information about the damage severity and the performance in order to distribute the limited recovery resources efficiently. Advances in sensor technology, wireless communication, and fast data processing capacities, combined with advancements in vibration techniques and damage identification/localization algorithms, would allow an efficient seismic impact assessment tool for large stock of infrastructure components (Büyüköztürk and Günes, 2002).

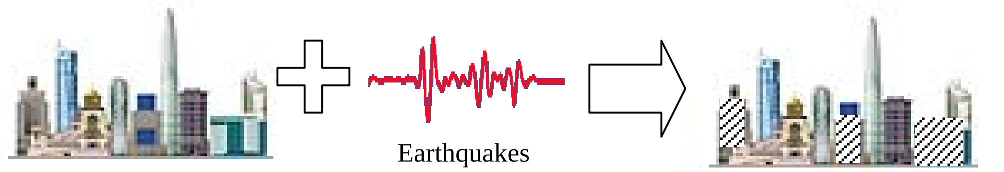


Figure 1. Formation of Damaged Civil Infrastructure

Conventional approaches of damage assessment based on visual inspections inch-by-inch over the structure are inefficient, costly and unreliable. Other discrete condition assessment approaches, or event-driven condition evaluation, only provide snapshot observations on the structure with respect to specific times. Estimation of long-term structural behavior and the changes in its integrity based on finite discrete measurements may be inadequate, especially for problems like fatigue and creep. Therefore, structural health monitoring (SHM) has been introduced as a global approach for damage detection. Damage detection, or damage identification, implies the determination of damage severity, damage location, and damage orientation. Damage detection is based on measurement parameters, such as acceleration, displacement, deflection, strain, rotation, temperature, and relative humidity. Importance and potential advantage of SHM are exemplified by worldwide monitoring activities (Mufti, 2002) (Inaudi, 2000) (Sumitro *et al*, 2001) (Thomson *el al*, 2001). Critical civil infrastructure systems, such as long-span bridges, have been instrumented to monitor their performance under operation loads and environmental influences, as well as in hazardous events.
In what follows, an overview is given of the components of a SHM system followed by discussion of damage detection algorithms. Relation of structural sensing to seismic impact assessment is then discussed and an example of an ongoing instrumentation project is given. Role of SHM in seismic impact assessment through an integrated, conceptual impact simulator is also discussed.

OVERVIEW

Structural health monitoring (SHM) system is a tool to detect, measure, and record the field performance of structure, in terms of the parameters relevant to the state of structure and its environment, such as acceleration, displacement, strain, temperature, and humidity. SHM helps in defining the critical response of a structure to track and evaluate the symptoms of operational incidents, anomalies, deterioration, and damage that may impact structure's serviceability and safety. SHM can achieve the goals of an effective infrastructure maintenance methodology by providing the following information:

- Knowledge on static and dynamic behaviors of civil infrastructure,
- Improved understanding of design assumptions (boundary conditions),
- Study of the performance of different design philosophies and construction techniques,
- Development and tuning of the corresponding numerical model (FEM),
- Evaluation of short-term impact due to natural hazardous events (earthquakes),
- Evaluation of short-term impact due to manmade activities (rehabilitation, retrofitting, expansion),
- Determination of actual load carrying capacity and monitoring of long-term deformation due to operational loading,
- Monitoring of long-term deterioration due to physical aging,
- Investigation of the pattern of environmental variation and influence on the measurements (temperature fluctuation),
- Providing health-related information for civil infrastructure management as a basis to optimize the allocated resources,
- Extending structures' service life by adequate repair and retrofit efforts.

SHM is a multidisciplinary system integration approach, which involves (1) sensing technology, (2) power technology, (3) communication technology, (4) storage technology, (5) signal processing, and (6) health evaluation algorithm. A schematic illustration is given in Fig. 2.

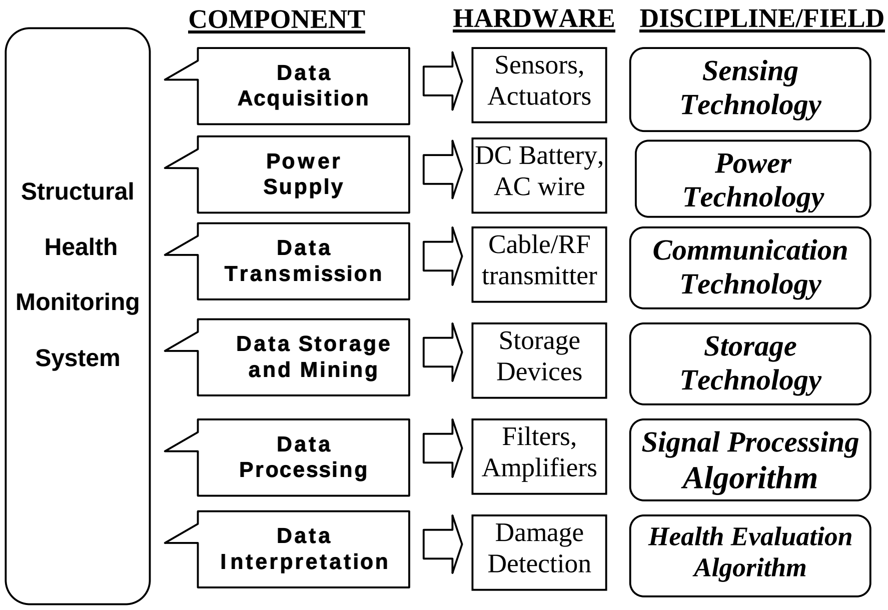


Figure 2. Typical Components of Structural Health Monitoring Systems

Data Acquisition:

Sensing technology is basically the methodology and techniques developed for detecting disturbance, variance, and change in physical systems. "Sensing" represents "being aware of some existence", thus the existence must be physically detectable and there must be corresponding devices (transducers) to receive the signal from the existence following specific physical laws. It is worth to point out that the format of the signal we receive at the user end relies on not only the physical form the source emits, but also the characteristic the sensor possesses.
A broad definition of sensing technology can be considered involving sensor technology and instrumentation technology. Sensor technology is mainly considered as the manufacturing techniques for sensors (transducers). Sensors are the first essential component in the monitoring chain and are responsible for the accuracy and reliability of the measurement. Instrumentation technology includes data recording, data representation, and sensor network (Morris, 1993).

Definition of Sensors:

In general, a sensor is a "device for sensing a physical variable of a physical system or an environment" (Iyengar *et al,* 1995). A sensor is any device that responses to the physical variable of interest that is being monitored. A sensor can be designed to be single-functional or multi-functional. Modulus design integrates different types of sensor onto a multi-functional sensing unit to fit the need of various measurements.

Classification of sensors:

Sensors are classified by the capability of measuring desired physical and/or chemical variables. Examples of some measurable quantities are:

- Mechanical quantities: Displacement, length, volume, location, level; Velocity, acceleration; Pressure, force/torque, twisting, weight; Strain; Rotation; Distortion; Flow.
- Thermal quantities: Temperature; Heat.
- Electromagnetic/optical quantities: Voltage, current, frequency phase; Visual/images, light; Magnetism.
- Chemical quantities: Moisture, pH value.

Attributes of sensors:

The design of sensing systems depends on the choice of sensors whose attributes match the requirements of the application. Attributes of sensors, as summarized below, can be used as a criterion for sensor selection and performance evaluation (Iyengar, *et al,* 1995).

- Accessibility: The sensing capability of sensors associated with specific precision in certain range.
- Dimension of Variables: The number of dimensions of physical variables.
- Size: The physical volume of sensors.
- Operating Range: The designed range of sensors for optimal performance.
- Data Format: The measuring feature of data in time; continuous or discrete/analog or digital.
- Sensitivity: The minimum magnitude or change in the environment sensors can detect.

- Intelligence: Capabilities of on-board data processing and decision-making.
- Active versus Passive Sensors: Capability of generating vs. just receiving signals.
- Physical Contact: The way sensors observe the disturbance in environment.
- Operating Principle: Embedded technologies that make sensors function, such as electro-optics, electro-magnetic, piezoelectricity, active and passive ultraviolet.

Power Supply:

Sensors are driven by power. Issues of power supply involve power generation (source of energy), power transmission (path of energy) and power storage (location of energy). In the design of power supply for distributed sensors network, power transmission and power storage are the main concerns. For self-powered sensors, power generation (or power harvesting) mechanism and power storage are critical issues. In general, design issues for power supply can be organized into the following points:

- Power Generation: External powered or self-powered.
- Power Transmission: Capacity of wires.
- Power Storage: Capacity of batteries.
- Power Consumption: Energy-consumption efficiency, power-efficient and low power technology.

Many of the sensors used in health monitoring systems are powered by lead wires, especially for purpose of experiment in laboratory. At present, many of the sensors in practice use batteries to guarantee the stability of power supply. For wireless sensors, the use of battery seems to be inevitable. When batteries are used, durability of batteries will be a concern.

Data Transmission – Communication Technology:

The purpose of data transmission is to deliver information through processing data in different formats. Fundamental types of data or signals are analog and digital. For analog systems, data are stored in form of the combination of oscillating waves of different frequencies. For digital systems, data are stored inside a storage device, in the form of binary digits or bits, and transmitted along the communications path between sensors and computers using electrical signals. Data can be carried by substantial wire or radio transmission. On a wire path, signal propagation is a flow of electrical current. Radio transmission is accomplished by emitting an electrical signal that propagates as an electromagnetic wave.

Available physical data transmission paths are:

- Wire pairs, cables, and coaxial cables
- Microwave
- Optic Fibers

Design issues for data transmission are type of transmission (wire pairs, cables, microwave, optic fibers), transmission rate (bits per second, bps), transmission bandwidth (frequency bandwidth, Hz), and transmission standard (interface standards: RS232, RS449).

Data Storage and Mining – Storage Technology:

In monitoring civil infrastructure through the use of sensors, the amount of measurement may increase rapidly based on the following main reasons:

- Typical dimensions of civil infrastructure potentially require vast amounts of sensors to achieve necessary deployment density (large amount of data from sensors in space)
- Long-term behavior of civil infrastructure needs continuous monitoring to depict the detail of time-variant phenomenon (large amount of data from measurements in time)

The first reason relates to the issue of sensor deployment and the second relates to frequency of monitoring. Although the current sensor deployment philosophy tends to minimize the required amount of sensors to capture the desired vibration modes and local behavior, redundant measurement may still be necessary since the response of structures due to some vibration events, such as earthquake, will not be repeated. The current technological advances in database management can be integrated into the development of SHM.

Data Processing – Signal Processing Algorithm:

The mission of data processing involves two main tasks:

- Signal De-noising
- Signal Compression

In an open environment, which is the case of real world, unavoidable and unidentified signals will be measured as well while recording the desired signals. Unavoidable signals are attributed to environmental influences, such as temperature fluctuation. This type of signals is rather pattern-based and can be filtered out after the recognition of their pattern of changes. Unidentified signals are of random nature and are usually considered as white noise (zero mean and unity

standard variation) Signal de-noising plays a crucial role in detecting the onset of incipient damage based on small disturbances (changes) in measurement. The de-noising task is achieved by designing filter banks.
With the increasing number of sensors and the characteristic of continuous reading, the amount of possible data record rapidly grows and the need to shrink the size of data becomes explicit. This is especially helpful during data transmission when wireless communication technology is adopted. Novel signal processing technology, such as wavelet theory, exhibits the validity of bridging signal processing and structural health monitoring.

Data Interpretation – Damage Detection Algorithm:

The core knowledge of health monitoring is the interpretation of measurement. SHM aims at the investigation of structural integrity, as related to the changes in structural properties (mass, stiffness, damping), through monitoring the changes in static and dynamic responses of the structure. Damage detection or damage identification is the kernel technique of SHM providing the interpretation and explanation of the measurement with respect to the state of the structural health. In what follows, some principles will be given.

DAMAGE DETECTION ALGORITHM

Various damage detection algorithms have been proposed based on different physical and mechanical principles. Anomalies in stiffness, inertial characteristics, stress level, and damping level can be estimated based on the collected data by distributed sensor networks. In general, damage detection algorithms can be classified into two categories: modal-based and non-modal-based. Modal-based algorithms are derived from the formation of modal analysis in structural dynamics. The changes in structural properties are represented in terms of modal parameters by definition. Non-modal-based algorithm depicts the change in structural properties in terms of structural response, such as interstory drift.

As a basis for the evaluation of the structure, any changes in the structural properties based on measured data must first be reconstructed. There are various modal-based methods to relate the observed data to the structural properties. These are called modal parameter identification or realization methods. The categorization of modal parameter identification methods is as follows:

- Indirect Time Domain Analysis Method: The Eigensystem Realization Algorithm (ERA) (Juang and Pappa , 1985, 1986) (Juang, 1987)
- Direct Time Domain Analysis Method: The Autoregressive Moving-Average method (ARMA) (Gersch and Martinelli, 1976) (Gersch and Brotherton, 1982)
- Indirect Frequency Domain Analysis Method: The Complex Exponential Frequency Domain method (CEFD) (Schmerr, 1982)
- Direct Frequency Domain Analysis Method: The Identification of Structural System Parameters method (ISSPA) (Link, 1986)

Depending on the characteristic of input and output information, identification methods may be categorized into four types: Single-Input Single-Output (SISO), Single-Input Multi-Output (SIMO), Multi-Input Multi-Output (MIMO), and Multi-Input Single-Output (MISO). Measurements are the input data and structural properties are the output data. Identification based on MIMO and SIMO take advantage of distributed sensors leading to a more detailed description of the structural property. Among current methods, the Eigensystem Realization Algorithm (ERA) is widely used and therefore introduced below.

Modal Parameter Estimation Methods – Eigensystem Realization Algorithm:

Developed by Juang and Pappa (1985, 1986), ERA allows one to select finite reliable measurements from infinite possible measurements. The objective of the realization process is to obtain the minimum order of the state-space formulation that can still represent the dynamic behavior of the structure.
Consider an N degree-of-freedom damped structure obeying the equation of motion:

$$M\ddot{x} + C\dot{x} + Kx = f(x,t) \tag{1}$$

where M, C, K are the mass, damping, and stiffness matrices of $n\times n$, respectively; $\ddot{x}$, $\dot{x}$, x are the acceleration, velocity, and displacement vectors at measured locations. They are all functions of time, t. $f(x,t)$ is the input force vector. A state vector is defined.

$$\{u(t)\}_{2n\times 1} = \begin{Bmatrix} x(t) \\ \dot{x}(t) \end{Bmatrix}_{2n\times 1} \tag{2}$$

Coefficient matrices are also defined.

$$[A]_{2n\times 2n} = \begin{bmatrix} 0 & I \\ -K/M & -C/M \end{bmatrix}_{2n\times 2n} = \begin{bmatrix} a_{ij} \end{bmatrix}_{2n\times 2n}, \quad [B]_{2n\times m} = \begin{bmatrix} 0 \\ F/M \end{bmatrix}_{2n\times m}$$

$$\{f(x,t)\}_{n\times 1} = [F]_{n\times m} \cdot \{\delta(t)\}_{m\times 1} \tag{3}$$

where $\{\delta(t)\}$ is the input force vector at m locations, $[F]$ is the input coefficient matrix. The equation of motion in the state space can be rewritten as:

$$\{\dot{u}(t)\}_{2n\times 1} = [A]_{2n\times 2n} \cdot \{u(t)\}_{2n\times 1} + [B]_{2n\times m} \cdot \{\delta(t)\}_{m\times 1} \tag{4}$$

State vector can be related to the displacement vector observed from p locations through a transformation matrix T:

$$\{x(t)\}_{p\times 1} = [T]_{p\times 2n} \cdot \{u(t)\}_{2n\times 1} \tag{5}$$

Through solving Eqn. (4) and discretizing the time vector by taking $t_0 = h\cdot\Delta t$ and , state vector is evaluated

$$\{u(h+1)\}_{2n\times 1} = [\bar{A}]_{2n\times 2n} \cdot \{u(h)\}_{2n\times 1} + [\bar{B}]_{2n\times m} \cdot \{\delta(h)\}_{m\times 1} \tag{6}$$

$$\text{where } [\bar{A}]_{2n\times 2n} = e^{[A](t-t_0)}, \qquad [\bar{B}]_{2n\times 2n} = -\int_0^{\Delta t} [\bar{A}]_{2n\times 2n} \cdot [B]_{2n\times m} \cdot ds' \tag{7}$$

and s' is a variable defined between 0 and Δt for any $t \geq t_0$ while assuming $\{\delta(t)\}$ is constant over the time interval $h\Delta t \leq t \leq (h+1)\Delta t$ and Δt is small. Meanwhile, Eqn. (5) takes the form of

$$\{\bar{x}(h)\}_{p\times 1} = [T]_{p\times 2n} \cdot \{u(h)\}_{2n\times 1} \tag{8}$$

The realization algorithm is actually the process that determines matrices $[T]$, $[\bar{A}]$, and $[\bar{B}]$. Matrix $[T]$ denotes the mapping from the state vector, $\{u(t)\} = \{u(t, x(t), \dot{x}(t))\}$, to the displacement vector, $\{\bar{x}(t)\}$. Determination of matrices $[\bar{A}]$ and $[\bar{B}]$ will lead to $[A]$ and $[B]$, which are consisted of structural property matrices ([M], [C], [K]). Due to the inverse-problem nature, the solution is non-unique. Energy-based convergence criterion is required during the iteration process with respect to discretized time series responses. There is infinite number of realizations, theoretically. ERA allows one to choose the better responses with less influence of noise, which are the locations of sensors and the number of sensors, p. With the aid of realization algorithms, structural properties can be reconstructed from field measurements.

Modal-Based Damage Detection Algorithms – Modal Analysis:

Modal analysis is primarily a tool for modeling dynamic characteristics of structures. Dynamic characteristics of engineering structures are usually represented by modal frequencies, damping ratios, mode shapes, and their derivatives. Modal analysis methods can be categorized into two types with respect to the attribute of variable domain; frequency domain and time domain. Frequency domain analysis relies on measured frequency-response-function (FRF) data. It extracts vibration modes from measured FRF data and derives modal data from FRF data. Peak-picking method, circle fit method, inverse FRF method, least-square method, and Dobson's method belong to frequency domain analysis. Time domain analysis uses response data either from known excitation sources or from ambient excitation. With known excitation, the measurement conditions are similar to the frequency domain analysis. Time domain analysis extracts the modal parameters from the reduction of the impulse response information from measured time response or the FRF data, based on the fact that the FRF and the impulse response function (IRF) are a Fourier transform pair. To depict the relationship between changes in structural properties and modal parameters, sensitivity analysis is usually performed. Observation on modal frequency changes, mode shape changes, and modal damping changes will be discussed below.

Consider formulation of a structure as a multi-degree-of-freedom (MDOF) system in terms of the mass, damping, and stiffness matrices representing structural properties:

$$[\mathrm{M}]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{F(t)\} \tag{9}$$

Among these structural properties, the form of velocity-dependent damping implies the assumption of viscous damping. The use of viscous damping is for mathematical convenience in formulation (Crandall, 1970) (Jeary, 1997). Further discussion of the use of damping model is beyond the scope of this paper. Eqn. (9) can be rewritten as

$$\left(s^2\left[M\right]+s\left[C\right]+\left[K\right]\right)\left\{\bar{X}\right\}e^{st}=\left\{F(t)\right\} \tag{10}$$

where $\left\{x\right\}=\left\{x(t)\right\}=\left\{\bar{X}\right\}e^{st}$. Eqn. (10) can be considered as the transform of external loading $\left\{F(t)\right\}$ to structural response $\left\{\bar{X}\right\}$. Through the analysis of eigen problem, eigenvalues, $\left[\Lambda\right]$, and eigenvectors, $\left[\Phi\right]$, are evaluated.

The construction of mass [M], damping [C], and stiffness [K] matrices is the representation of spatial model of structures and the modal model of structures is denoted by eigenvalue $\left[\Lambda\right]$ and eigenvector $\left[\Phi\right]$ matrices. A general form relating the external loading (input) to structural response (output) is

$$\left\{x(t)\right\}=\left\{\bar{X}\right\}e^{st}=\left[H(\omega)\right]\left\{F(t)\right\} \tag{11}$$

and $\left[H(\omega)\right]=\left\{s^2\left[M\right]+s\left[C\right]+\left[K\right]\right\}^{-1}$; $\left[H(\omega)\right]$ is called the response function. A schematic illustration depicting the interrelation of these models is provided in Fig. 3.

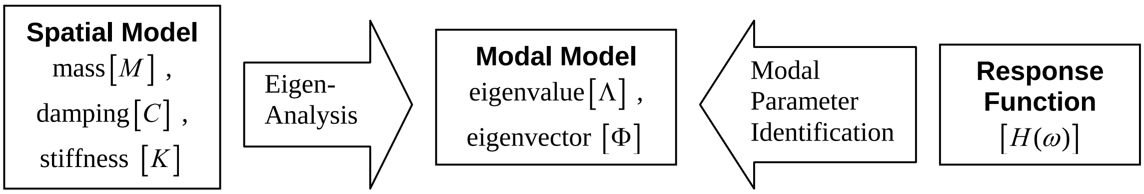


Figure 3. Interrelation of Spatial and Modal Models

Modal-based damage detection algorithms focus on the change in modal parameters and correlate it to the change in structural properties representing by spatial model.

Modal frequency changes:

The eigenvalue equation of an undamped system is

$$\left(K-M\cdot\lambda\right)\cdot\phi=0 \tag{12}$$

where *K* is the stiffness matrix, *M* is the mass matrix, λ is the eigenvalue of the system, and ϕ is the mode shape vector. Consider small changes in the stiffness matrix and mass matrix, denoted by δK and δM , respectively. Eqn. (12) becomes

$$\left[\left(K+\delta K\right)-\left(M+\delta M\right)\left(\lambda+\delta\lambda\right)\right]\cdot\left(\phi+\delta\phi\right)=0 \tag{13}$$

For a non-trivial solution of ϕ , $K-M\cdot\lambda=0$. Pre-multiplying Eqn. (13) by ϕ^T and rearranging, one obtains an expression for $\delta\lambda$:

$$\delta\lambda=\frac{\phi^T\cdot\delta K\cdot\phi+\phi^T\cdot\delta K\cdot\delta\phi-\lambda\left(\phi^T\cdot\delta M\cdot\phi+\phi^T\cdot\delta M\cdot\delta\phi\right)}{\phi^T\cdot M\cdot\phi+\phi^T\cdot\delta M\cdot\phi+\phi^T\cdot M\cdot\delta\phi+\phi^T\cdot\delta M\cdot\delta\phi} \tag{14}$$

Above, δK , *M*, and δM are matrices, ϕ and $\delta\phi$ are vectors, λ and $\delta\lambda$ are scalars with respect to each mode. Eqn. (14) is the general form of $\delta\lambda$ for an undamped system. If the change in mass is prohibited, which means $\delta M=0$, Eqn. (14) can be reduced to

$$\delta\lambda=\frac{\phi^T\cdot\delta K\cdot\phi+\phi^T\cdot\delta K\cdot\delta\phi}{\phi^T\cdot M\cdot\phi+\phi^T\cdot M\cdot\delta\phi}=\delta\left(\omega^2\right) \tag{15}$$

$$\delta\omega=\frac{1}{2\omega}\cdot\frac{\phi^T\cdot\delta K\cdot\phi+\phi^T\cdot\delta K\cdot\delta\phi}{\phi^T\cdot M\cdot\phi+\phi^T\cdot M\cdot\delta\phi} \tag{16}$$

For practical purposes, an approximate form is preferable. By neglecting the second-order terms produced by δK , $\delta\lambda$ and $\delta\phi$, an approximate form is obtained.

$$\delta\lambda = \frac{\phi^T \cdot \delta K \cdot \phi}{\phi^T \cdot M \cdot \phi + \phi^T \cdot M \cdot \delta\phi} \tag{17}$$

$$\delta\omega = \frac{1}{2\omega} \cdot \frac{\phi^T \cdot \delta K \cdot \phi}{\phi^T \cdot M \cdot \phi + \phi^T \cdot M \cdot \delta\phi} \tag{18}$$

Assuming no mode shape change, Eqn. (17) can be further reduced to the results proposed by Cawley and Adams (1979). $\delta\omega$ is the change in the natural frequency, δK is the change in structural stiffness, M is the mass matrix, and ϕ denotes the mode shape vector. Mass and stiffness matrices may be reconstructed from a realization algorithm, such as ERA. Mode shape vector ϕ and natural frequency change, $\delta\omega$, can be calculated from the measurements through frequency analysis. Determination of the damage location requires discretization of the stiffness matrix.
Since stiffness is associated with specific type of loading (axial, flexural, torsional), the change in stiffness might be insensitive to specific damage if incorrect type of stiffness matrix is used. The influence of assumed damage type on different stiffness values can be realized through numerical simulation and experiments.

The features of the modal frequency change method are listed below, which can also be considered as the common characteristics of other frequency-based damage detection algorithms.

1. The existence of damages can be assessed through the changes in the natural frequencies.
2. Without dividing the structure into parts, the solution of damage locations is not unique.
3. The content of damages is investigated through the influence of damages on different types of stiffness (axial, flexural, torsional).
4. In practice, only the natural frequencies of lower modes are possible for evaluation since higher modes are difficult to be excited.

Qu (2000) presented two modal acceleration methods, based on hybrid expansion, power series expansion and modal superposition of the dynamic flexibility matrix, to reduce the modal truncated errors of the frequency response functions (FRF) and their sensitivities. Some researchers proposed different approaches to locate damage using the frequency-based algorithm, sensitivity between frequency changes and member stiffness changes (Stubb *et al,* 1990) (Stubbs and Osegueda, 1990), damage index method (Stubb *et al,* 1992), inverse approach (Nakris, 1994), auto-regressive moving average (ARMA) model (Brinker *et al*, 1995), damage location assurance criterion (DLAC) method (Messina *et al*, 1996).
From the previous work using frequency changes as the indicator of damage detection, it is considered that the relatively low sensitivity of frequency changes to damage requires precise measurements. Since modal frequencies are a global description of structures, generally it cannot provide spatial information (location, orientation) of damage if separation of structure is not performed. It is only in higher modes modal frequencies can express local changes, which is the nature of incipient damage (Doebling *et al,* 1996).
However, exciting higher modes of civil structures experimentally is still very difficult. Meanwhile, high modal density and low participation factors of higher modes increase the difficulty of identifying damage using modal frequencies (Farrar and Doebling, 1999). Significant changes in modal frequencies alone do not imply the existence of damage since frequency changes due to changes in ambient conditions have been measured for both concrete and steel bridges within a single day. It would be necessary for a modal frequency to change by about 5% for damage to be detected with confidence. Changes in modal frequencies can be attributed to support failure, crack propagation, shear failure, and overload causing internal damage. From the research findings proposed by other researchers, it is suggested that detection of damage using frequency measurements might be unreliable when the damage is located at regions of low stresses. The effect of environmental factors on changes in modal frequencies is small as reported (Salawu, 1997).

Mode shape changes:

Mode shape change or modal strain energy methods are believed to be the promising approach to locate incipient damages. It can be verified from the derivation of $\delta\phi$ as shown below.

Recall Eqn. (13)

$$\left[\left(K + \delta K\right) - \left(M + \delta M\right)\left(\lambda + \delta\lambda\right)\right] \cdot \left(\phi + \delta\phi\right) = 0$$

With $K - M \cdot \lambda = 0$, it yields

$$\delta\phi = \frac{\delta K \cdot \phi - \lambda \cdot \delta M \cdot \phi - \delta\lambda \cdot M \cdot \phi - \delta\lambda \cdot \delta M \cdot \phi}{-\delta K + \lambda \cdot \delta M + \delta\lambda \cdot M} \tag{19}$$

Available comparison techniques for mode shapes are more or less based on the orthogonality conditions, such as the Modal Assurance Criterion (MAC), the Normalized Cross Orthogonality (NCO), and the Normalized Modal Difference (NMD). The most common means of comparing two modes is probably by the Modal Assurance Criterion, which is defined as follows: (Allemang and Brown, 1982)

$$\mathrm{MAC}\left(\{\phi_d\}_i,\{\phi_u\}_j\right)=\frac{\left|\{\phi_d\}_i^T\cdot\{\phi_u\}_j\right|^2}{\left(\{\phi_d\}_i^T\cdot\{\phi_d\}_i\right)\cdot\left(\{\phi_u\}_j^T\cdot\{\phi_u\}_j\right)} \tag{20}$$

where $\{\phi_d\}_i$ and $\{\phi_u\}_j$ are two mode shape vectors from damaged and undamaged structures. The advantage of the MAC is that the use of reduced structural experimental matrices is sufficient. Coordinate complete experimental data are not required. When two reduced mode shape vectors are identical, it will not result in a true identity matrix due to the omission of the structural matrices. Even though the MAC is highly effective for many structures, it has been unreliable in the correlation of reduced local modes.

Numerical examples

A three DOF undamped mass-spring system is described in Figure 4.

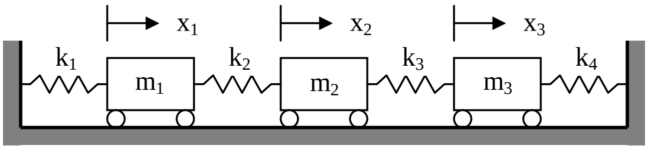


Figure 4. An Undamped 3-DOF System

Assume $m_1 = m_2 = m_3 = m$ and m = 100 kg in mass matrix. Also, assume $k_1 = k_2 = k_3 = k_4 = k$ and k = 10^5 kg/m. The natural frequencies and mode shape vectors are evaluated through solving eigenvalue problem. The natural frequencies are $\omega_1 = 24.203$ rad/sec, $\omega_2 = 44.721$ rad/sec, $\omega_3 = 58.431$ rad/sec.

Single stiffness-type damage is introduced by simulating the damage as the reduction of stiffness for one specific structural member. The reduction fraction of stiffness may vary from 0% to 100%. It is desired to know how the changes in local stiffness values affect the changes in global natural frequencies, associated with the changes in the curvature pattern of stiffness reduction.

– Effect of local stiffness reduction on global natural frequency changes

To investigate the effects of the magnitude of local stiffness changes on global natural frequencies, a quantitative numerical study is performed. For the example of member 1, the damage is simulated as stiffness reduction at member one (k1). The quantitative relation between local stiffness reductions in member 1 (k1) and global natural frequency changes are shown in Fig. 5.

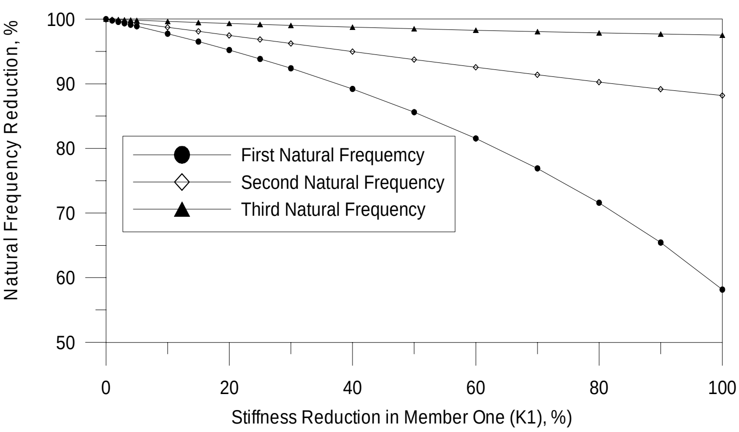


Figure 5. Local Stiffness Reduction (k1) versus Global Natural Frequency Changes

From Fig. 5, it is seen that the local stiffness reduction (k1) produces different influences on different modes. The first global natural frequency decreases rapidly with reduced local stiffness. A maximum of 40% reduction in the first natural frequency is observed with 100% local stiffness decrease. Compared to the first, the second and the third natural frequencies decrease at a relatively reduced rate as the local stiffness decreases. Influences of local stiffness reduction on global natural frequency change are distinguishable as long as the measurement of natural frequencies is available.

– Effect of the local stiffness reduction on the rate of global natural frequency changes
It is also of interest to know how the rate of global natural frequency changes is influenced by the local stiffness reduction. As shown in Fig. 6, the trend of global natural frequency reduction varies from mode to mode. For the first mode, the rate of natural frequency change increases with decreasing local stiffness. However, for the second and the third modes, the rate decreases with decreasing local stiffness.

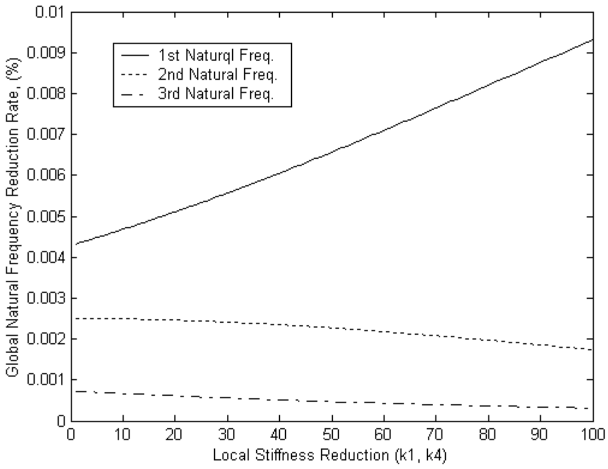


Figure 6. Reduction Rate of Global Natural Frequency vs.
Local Stiffness Reduction (k1, k4)

From the numerical results, it is also found that mode shape vector and its derivatives are sensitive to the location of damages. However, there are still some tradeoffs by using mode shape approach:
– It requires more measurements than frequency-based approach, at least two measurements to obtain slope data and three measurements to obtain curvature data.
– The resolution of mode shape data is generally inferior to that of frequency data.
– Mode shape data are more sensitive to the influence of noise.

Modal damping changes:

Damage detection algorithms based on changes in modal damping ratios are relatively less developed than stiffness and mode shape approaches. Damping is attributed to the phenomenon of energy dissipation in the form of attenuation of the structural response. Energy dissipation process or damping arises from many sources in a complex fashion physically. Fundamental understanding of the influence of damage on damping in structures has not been thoroughly established. Therefore, a simple and comprehensive mathematical model is not easily obtainable in capturing the essence of damping behavior.
In the dynamic analysis of structures, damping is usually specified in terms of modal damping ratios, and integrated into the modal-based equations of motion. As a general concept, at the structural level the existence of damage in structures will reduce stiffness and increase damping. Contribution of damping plays a critical role on the evaluation of load-carrying capacity when the behavior of structures enters the inelastic range.

Measurement of damping ratios can be achieved by the energy loss per vibration cycle, free vibration decay, half-power method (forced vibration), and logarithmic decrement (free vibration) (Tedesco *et al,* 1999). For undamped structures, changes in structural response are all attributed to changes in stiffness and mass. Formulation becomes simpler for such structures since the mass of most civil infrastructure systems remains constant or slightly changed within negligible range. For damped structures, the determination of damping matrix or modal damping coefficient is crucial.
Even though the actual mechanism of energy dissipation in real structures is closer to the hysteretic damping than to the viscous damping, the latter is widely adopted due to its efficiency and reliability. Mass-proportional, stiffness-proportional damping, Rayleigh damping, and Cauchy damping are several practical options to formulate the behavior of damping. Kawiecki (2001) applied arrays of surface-bonded piezoelectric transducers (PZT) for determining changes in damping levels as the basis of damage detection. Experimental results implied the feasibility of detecting damage through the change in damping levels. Oliveto and Greco (2002) examined and compared the influence of boundary conditions on different types of damping. They found that Rayleigh damping is the most general form that is independent of the boundary conditions.
From the previous findings, it is realized that reliable analysis results can be obtained only if an adequate choice of damping type is made. Velocity-dependent viscous damping has been intensively applied in the formulation of energy-dissipation mechanism based on its mathematical convenience.

Relation of Surface Measurement to Bending:

In this section, the determination of bending moment, and the effect of temperature variation are described by readings from an elongation-type sensor. Length variation of sensors depicts the elongation of the structure at finite locations with sensors deployed discretely. Average strain is calculated from the elongation data indicating local effect. It is assumed that the sensor is instrumented on the surface of the structure. Formulation of the relations of surface measurements to internal bending and deflections are provided in the following section.

Bending moment:

Consider a simply supported beam instrumented with one elongation-type sensor shown in Fig. 7. (x_i, y_i) denotes the location of sensor and l_i represents its original length. Bending moment at the location of the sensor is represented by

$$M = -EI\frac{d^2y}{dx^2}$$

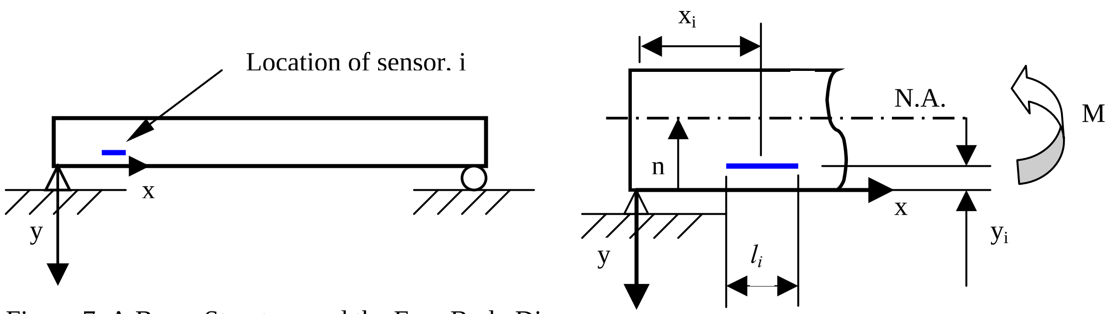


Figure 7. A Beam Structure and the Free-Body Diagram

where M is the bending moment, E is the Young's modulus, I is the moment of inertia, y is the vertical deflection, and x is the horizontal coordinate.

Average curvature κ_{ave} is calculated and related to the bending moment by Eqn. (21).

$$\kappa_{ave} = \frac{1}{r_{ave}} = \frac{d^2y}{dx^2} = -\frac{M}{EI} = \frac{l_j - l_i}{\left(n - y_j\right)l_i} \Rightarrow M = \left[\frac{l_i - l_j}{\left(n - y_j\right)l_i}\right]EI \qquad (21)$$

where l_i and l_j denote the lengths of the sensor before and after loading of the beam, respectively. Eqn. (21) gives the expression of bending moment at the location of the sensor by the change in length of the sensor. It is noted that the evaluated bending moment represents the average moment over the total length of the sensor.

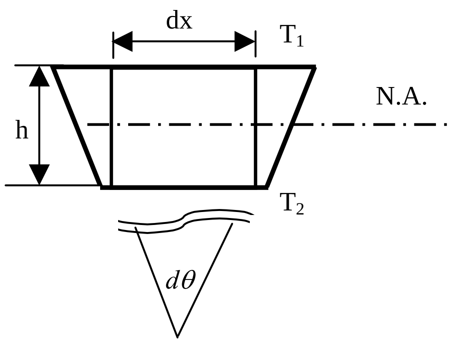


Figure 8. Cross-Sectional Deformation due to Temperature Variation

Temperature variation:

Structures are usually exposed to temperature changes. Temperature-induced internal actions may be evaluated using elongation-type sensors. Linear distribution of the temperature variation through the gross cross section is assumed. (Fig. 8)

$$hd\theta = \alpha\left(T_1 - T_0\right)dx - \alpha\left(T_2 - T_0\right)dx \Rightarrow \frac{d\theta}{dx} = \frac{\alpha\left(T_1 - T_2\right)}{h} = \frac{d^2y}{dx^2} = -\frac{M}{EI} \qquad (22)$$

The change due to temperature variation in length dx at the top of the cross section is $\alpha\left(T_1 - T_0\right)dx$ and at the bottom will be $\alpha\left(T_2 - T_0\right)dx$, where T_0 is the original temperature or baseline temperature, α is the thermal coefficient, h is the height of the cross section. The equivalent bending moment induced by the temperature variation can be related to the sensor measurements as follows:

$$M = -EI\frac{\alpha\left(T1 - T2\right)}{h} = -EI\left[\frac{l_j - l_i}{\left(n - y_j\right)l_i}\right] \Rightarrow \frac{\alpha\left(T1 - T2\right)}{h} = \frac{l_i - l_j}{\left(n - y_j\right)l_i} \tag{23}$$

Combination of Eqn. (21) and (23) implies that, in the real world, the measurement can be affected by loading sources and “noise”. For instance, using a temperature-sensitive elongation-type sensor to measure the bending moment acting on the structure in a temperature-changing environment must deal with the problem of temperature-related noise. In general, such inverse-problems do not exhibit a unique solution. Hence, de-noising and signal filtering become a necessary component of SHM.

Reconstruction of Deflection by Surface Measurements:

Relation of measured deflections at finite locations to the computed deflection of a simply supported beam is described with a numerical example.

Finite element model:
A simply supported beam of 200 in span and the cross-sectional height of 20 in is considered. For the purpose of this example the measured values from sensors at discrete locations are assessed from a finite element solution. Therefore, a 2D finite element model is constructed and a plain stress analysis is performed. The assumed Young’s modulus, E, is 3,120,000 lb/in^2. A uniformly distributed loading of 5 lb/in/in is considered. Vertical deflections generated by FEM are illustrated in Fig. 9. In addition, curvatures at each node of the model are computed. The maximum vertical deflection at the mid-span of the beam, evaluated by FE analysis, is 0.0501 in.

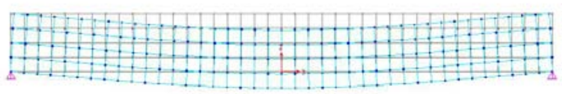


Figure 9. Deformed Shape of Simply Supported Beam

Two sensors are considered at the bottom of the structure. In order to explain the non-uniqueness of this inverse problem, four different configurations of sensor deployment are considered. Their locations are illustrated in Fig. 10.

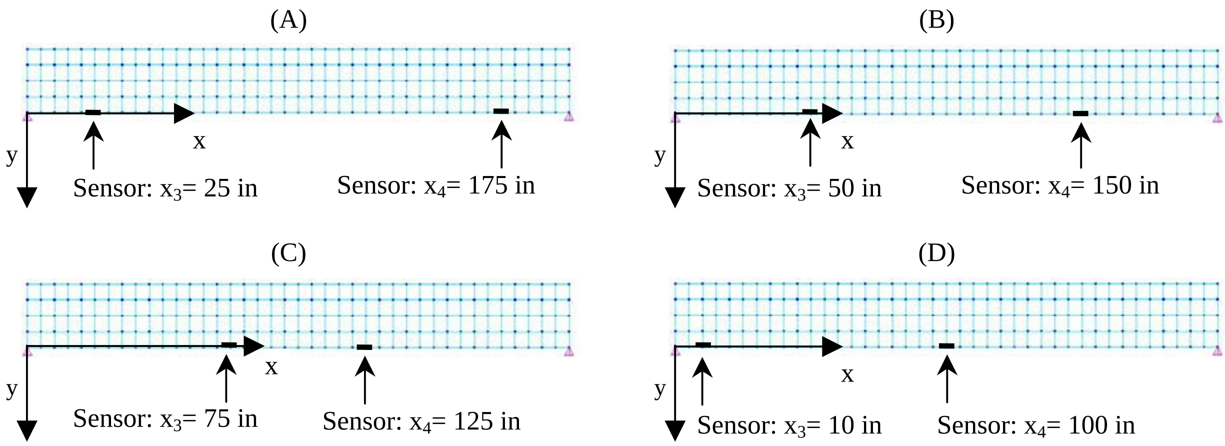


Figure 10. Configurations of Sensor Deployment

Reduced model:
Curvatures predicted from the FE solution are assumed as the measurements at the two sensor locations in the reduced model. The assumed polynomial expression of the deflection in the reduced model possesses four constants, as given below.

$$y\left(x\right) = a_1x^3 + a_2x^2 + a_3x + a_4 \tag{24}$$

By using the two boundary conditions (y(0)=0 and y(L)=0), and the two curvature measurements, Eqn. (24) can be evaluated. Considering

$$\frac{d^2y}{dx^2} = y'' = 6a_1x + 2a_2 \tag{25}$$

the four constants are solved:

$$\begin{Bmatrix} y_1 \\ y_2 \\ y_3'' \\ y_4'' \end{Bmatrix} = \begin{bmatrix} x_1^3 & x_1^2 & x_1 & 1 \\ x_2^3 & x_2^2 & x_2 & 1 \\ 6x_3 & 2 & 0 & 0 \\ 6x_4 & 2 & 0 & 0 \end{bmatrix} \begin{Bmatrix} a_1 \\ a_2 \\ a_3 \\ a_4 \end{Bmatrix}, \qquad \begin{Bmatrix} a_1 \\ a_2 \\ a_3 \\ a_4 \end{Bmatrix} = \begin{bmatrix} x_1^3 & x_1^2 & x_1 & 1 \\ x_2^3 & x_2^2 & x_2 & 1 \\ 6x_3 & 2 & 0 & 0 \\ 6x_4 & 2 & 0 & 0 \end{bmatrix}^{-1} \begin{Bmatrix} y_1 \\ y_2 \\ y_3'' \\ y_4'' \end{Bmatrix} \tag{26}$$

These constants are solved for given different configurations of the two sensors. Approximate vertical deflections for each scenario can be reconstructed using Eqn. (24). These approximate solutions are compared with the findings from finite element solution as shown in Fig. 11.

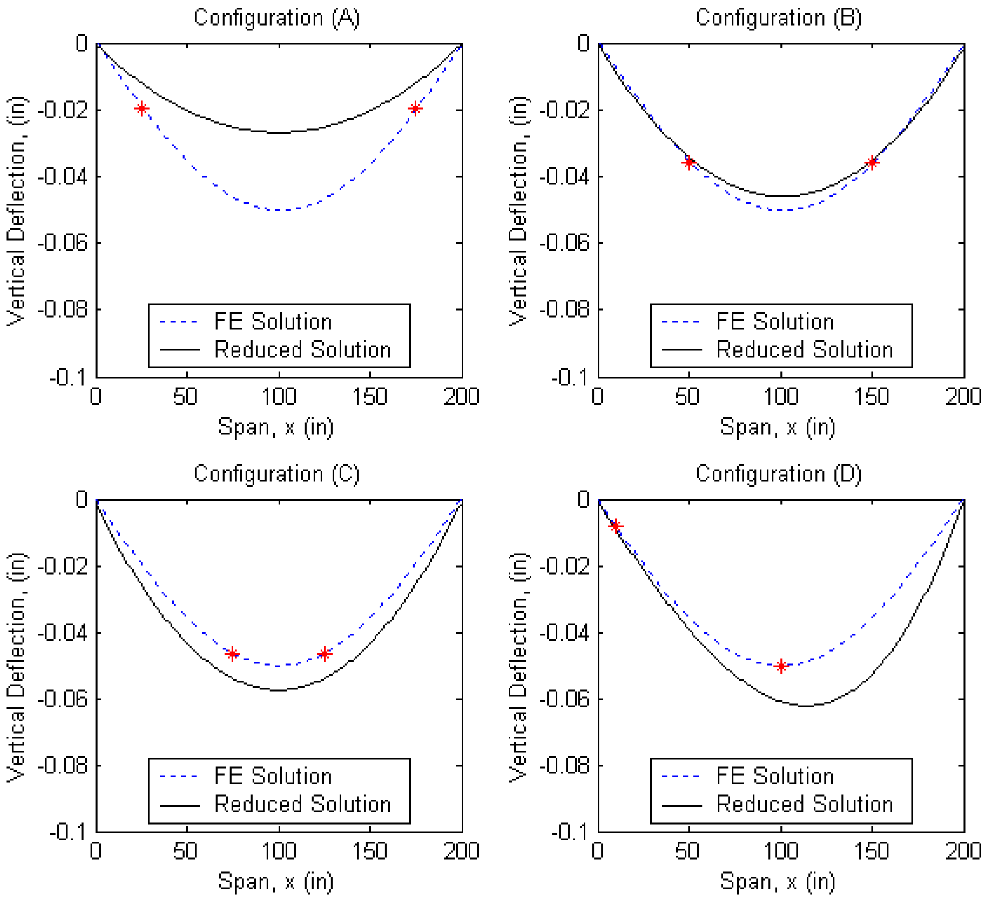


Figure 11. Comparison of FE Approach and Reduced Model Approach

From the estimations obtained by different measurements, it is realized that the solution of this inverse problem is not unique. Different input measurements will result in different estimations. The result also implies that the reliability of approximation depends on the location of the sensors. To compare the results from the reduced model with those from finite element solution, the MAC index and the deviation are applied as the quantitative criterion for evaluation. Results are as follows.

$$\mathrm{MAC}\left(\{\phi_{FEM}\},\{\phi_{RMA}\}\right) = 0.9982, \qquad \varepsilon_A = \sqrt{\left(\phi_{FEM} - \phi_A\right)^2} = 0.2250$$

$$\mathrm{MAC}\left(\{\phi_{FEM}\},\{\phi_{RMB}\}\right) = 0.9982, \qquad \varepsilon_B = \sqrt{\left(\phi_{FEM} - \phi_B\right)^2} = 0.0331$$

$$\mathrm{MAC}\left(\{\phi_{FEM}\},\{\phi_{RMC}\}\right) = 0.9982, \quad \varepsilon_C = \sqrt{\left(\phi_{FEM} - \phi_C\right)^2} = 0.0966$$

$$\mathrm{MAC}\left(\{\phi_{FEM}\},\{\phi_{RMD}\}\right) = 0.9861, \quad \varepsilon_D = \sqrt{\left(\phi_{FEM} - \phi_D\right)^2} = 0.1485$$

The information provided by the MAC and the deviation values indicate the correlation between the two vectors for each sensor configuration. It is found that configuration B describes the vertical deflection shape more accurately than the other three configurations. The result demonstrates the importance of selected sensor location.

STRUCTURAL SENSING AS A BASIS FOR SEISMIC IMPACT ASSESSMENT

Seismic Monitoring Systems:

Seismic monitoring is used for the purpose of disaster and emergency management, traffic control, and damage evaluation. Acceleration data measured by accelerometers can be used for evaluating the impact intensity of each earthquake event (Fig. 12). Being a seismic-active country, Japan established several national and local seismic monitoring systems for the purpose of detecting distribution of seismic intensity, damage assessment, loss estimation, and traffic control (Yamazaki, 2001). The use of seismic monitoring systems on underground pipeline systems may be mentioned as an example of disaster and emergency management. Secondary disasters induced by damaged gas utilities usually bring serious fatal casualties and property loss. Emergency shut-off function requires the information of abnormal pressure change in gas pipelines detected by pressure sensors. In the area of traffic management, speed limit control of roadways and operation shutdown of expressways need the input participation of sensors to render reliable traffic data and structural integrity information.

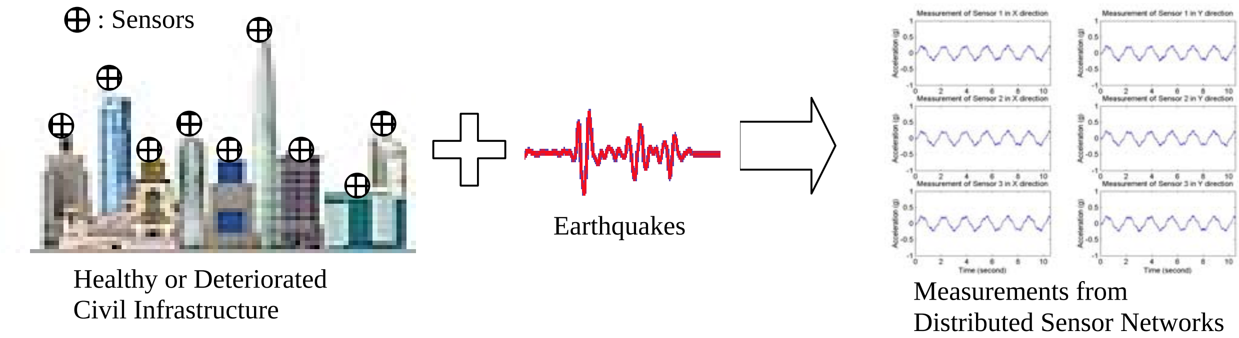


Figure 12. Measurements Collected from Distributed Sensor Networks

Response data provided by seismic monitoring systems can be used for the purposes of hazard analysis, local site effect analysis, structural vulnerability analysis, and estimation of loss. Effectiveness of strengthened and retrofitted structures, performance of supplemental energy dissipation and structural control, and efficiency of base isolation, can be verified through the measurements collected by seismic monitoring systems (Büyüköztürk and Günes, 2002).

The Flagpole Instrumentation Project

The flagpole project is one of several I-Labs within the I-Campus framework at the Massachusetts Institute of Technology (M.I.T.) involving the real-time instrumentation of a flagpole through the Internet. The project aims at the integration of knowledge from various disciplines, such as software engineering, sensor technology, signal processing, structural engineering and transportation engineering. This virtual laboratory is also used for educational purposes.

Three triaxial accelerometers and one thermocouple are installed on the flagpole structure (Fig. 13). The Flagpole WebLab provides real-time measurements of acceleration at points on the structure. Accelerations in two horizontal axes at three locations (labeled #1, #2 and #3) along the length of the flagpole are measured. The location of these points was chosen in order to be able to accurately represent the first three modes determined from a finite-element simulation of the flagpole. Additionally, a thermocouple was used to monitor the ambient temperature at the flagpole's base. Accelerations are uploaded into the database every minute and temperatures are uploaded once every 16 seconds. Three sensor modules consisting of triaxial accelerometers were mounted on the 15th of February 2002. The field-monitoring unit was activated on the 21st of March 2002 and all the software services went live on the 22nd. The modulus design of sensing device is shown in Fig. 14 and the components of the flagpole instrumentation project are illustrated in Fig. 15. Characteristics of accelerometer are listed in Table 1.

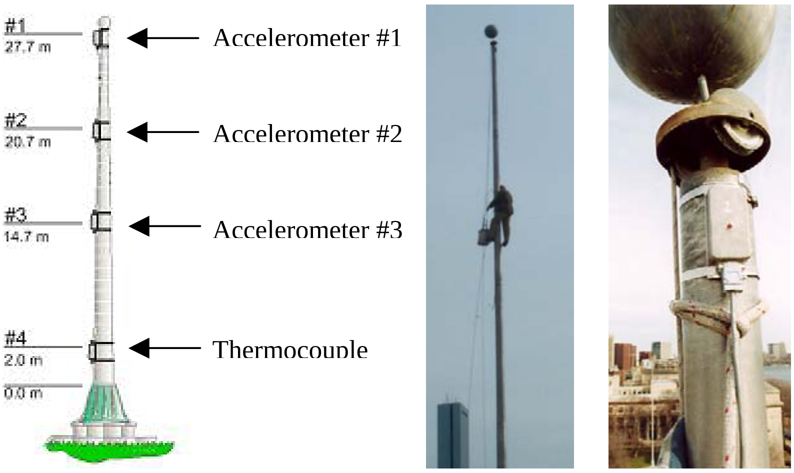


Figure 13. The Flagpole, Location of Accelerometers, and the Installation of the Sensors

Table 1. Characteristics of the Accelerometers

Sensitivity	1 V/g
Acceleration range	± 2g
Supply voltage	5 V
Zero acceleration voltage	2.5 ± 0.15 V

With this project it is planned to measure accelerations and obtain displacements, strains, stresses, and the temperature of the flagpole in ambient conditions. The goals include not only obtaining valid streaming data, but also setting up a foundation for further monitoring and decision support studies at M.I.T. The flagpole instrumentation project is closely related to the photovoltaic weather station project, which monitors environmental parameters such as wind speed, wind direction, rainfall, temperature, etc and makes them available on the Internet in real time.

The purposes of the measurements are

- To estimate damping, mode shape, noise level, wind pattern.
- To provide experimental data for active control algorithm.
- Fine-tuning parameters in numerical simulations (FEM model).
- Detection of distress in the structure.
- Real-time monitoring: To predict and prevent mishaps due to any structural failure.

For the purpose of this application, various components of the SHM system are provided as shown in Fig. 16.

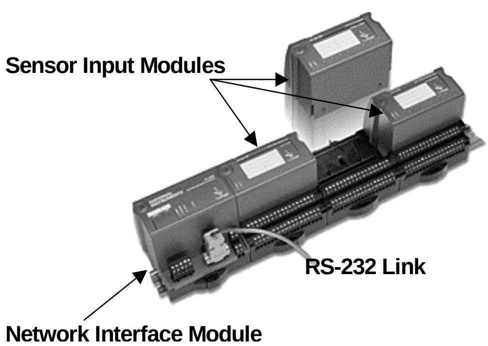


Figure 14 Modulus Design of Sensing Device (Amaratunga *et al,* 2002)

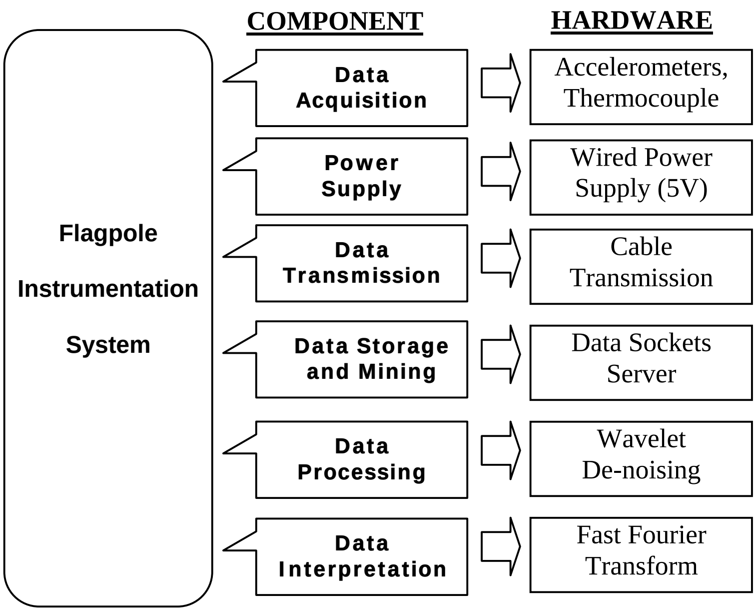


Figure 15. Components of the Flagpole Instrumentation Project

A Data Socket server is developed to handle all the communication between a publisher and a subscriber. The publisher first binds to a unique URL on Data Socket server (for example, dstp://fieldpoint.mit.edu/data) and starts publishing data. Various subscribers can then access the data published by reading data from the same URL. Therefore, the Data Socket handles many of the low-level implementation details and provides a convenient mechanism for transferring data among different programs. Fig. 16 illustrates this scenario, showing a publisher, implemented in LabWindows/CVI, sharing data with subscribers in many different platforms.

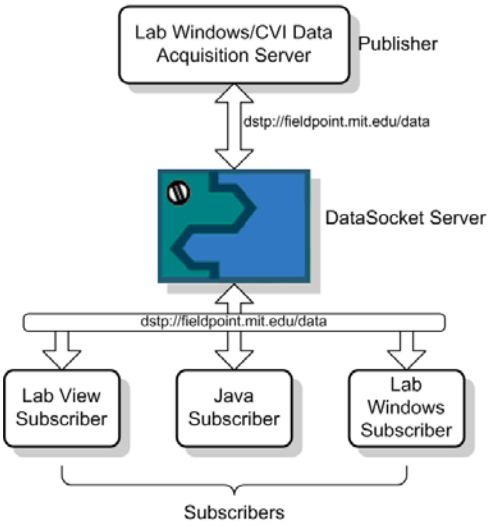


Figure 16. Exchanging Data Using Data Sockets
(Amaratunga *et al,* 2002)

Multiscale analysis – Wavelets:
With its success in image processing and compression, wavelet or ondelette has been considered one of the promising techniques in damage detection. The multiscale (also multilevel or multiresolution) feature enables wavelet to discover the local characteristics embedded in a signal, which can not be discovered by traditional analysis, such as Fourier analysis. Wavelets have been applied on image compression, edge detection, and partial differential equation (Williams and Amaratunga, 1994). Due to the efficient decomposition feature, the main characteristics of signals can be extracted and the "noise" part can be filtered out. The remaining part is the reconstructed result or the recognized result of the original signal. Fig. 17 shows the measurements collected from sensors in the form of a time-series signal. Voltage variations are sensed in these transducers and further translated into accelerations. Acceleration data collected from the three sensors in x- and y- axis are decomposed by wavelets. Fig. 18 shows the decomposition scenario by wavelets. Decomposed signal components, as well as the original signal, are illustrated in Fig. 19. De-noised signals are then processed through Fast Fourier Transform (FFT) (Fig. 20) to get the natural frequency data. Measured structural properties are further calculated. A corresponding finite element model is also constructed and the structural properties of the model are tuned to fit the measured data. Damage detection can be performed through comparing natural frequency changes and/or mode shape changes, which have been mentioned in the previous sections.

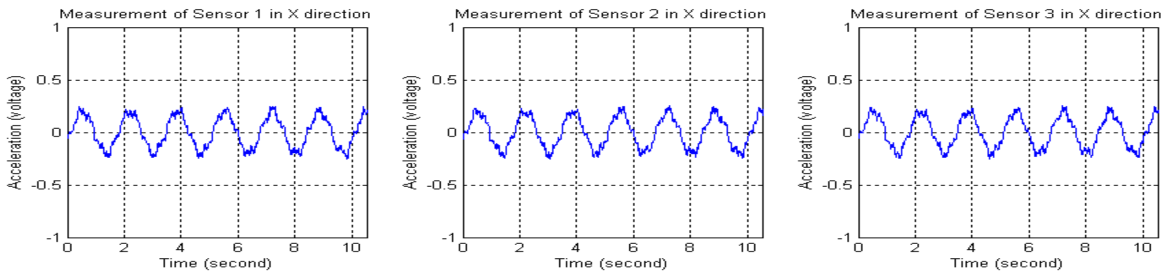
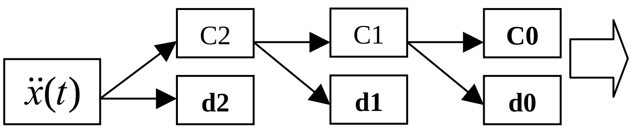


Figure 17. Sample Measurements from the Sensors



$$\ddot{x}(t)=c0+d0+d1+d2$$

Figure 18. Wavelet Decomposition Scenario

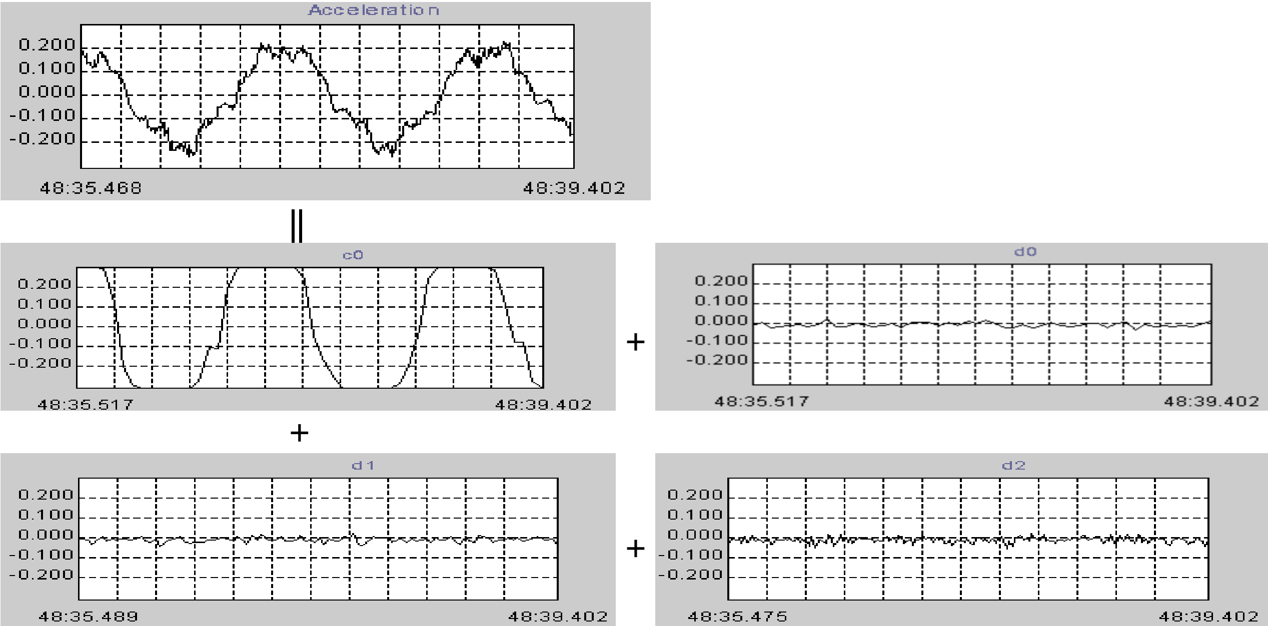


Figure 19. Wavelet Decomposition Scenario

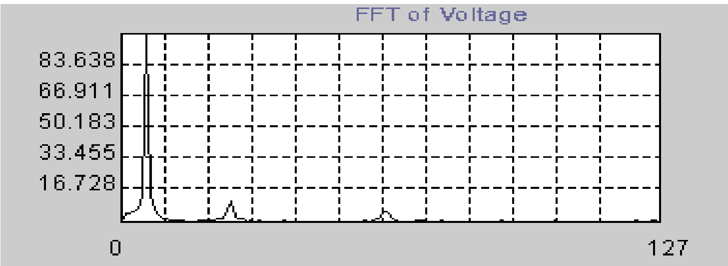


Figure 20. FFT of the Measurement from the Sensors

Structural properties are reconstructed in matrix form and the change in stiffness matrix is calculated.

$$[\delta K] = [K_{t1}] - [K_{t0}] = [K_d] - [K_u] \qquad (27)$$

where $[K_{t1}]$ is the stiffness matrix at time t1 and $[K_{t0}]$ is at time t0. Stiffness difference during t1 and t0 is denoted by $[\delta K]$. Through the approach shown above, the variation of stiffness is observed and monitored as the evaluation criterion of stiffness-type damage severity.

Seismic Impact Simulator:

Recent advances in computational resources and processing techniques have opened new horizons for effective risk assessment, hazard mitigation, and emergency management of existing civil infrastructure systems in urban areas. A research initiative at M.I.T. dealing with the development of large-scale disaster management system in urban areas focuses on earthquake impact simulation (Büyüköztürk *et al*, 2000). The objective of this system is to simulate a city's response to an earthquake by generating digital models of urban infrastructure. In that context, an integrated impact simulation environment called seismic impact simulator (SIS) is proposed.

Seismic impact, or physical impact of the earthquake, is represented in terms of damage severity or loss of capacity. Uncertain damage severity embedded in existing civil infrastructure requires structural investigation with an efficient strategy. Detailed inch-by-inch investigation may provide insight to the health profile of structures, but the required time and human effort make this option unfeasible in view of the vast amount of building stock. As a global approach, SHM in the form of distributed sensor networks can be used on critical civil infrastructure to monitor the change in structural integrity. Under service conditions, responses of structures are recorded and the influence of operational loading on structural health is evaluated. After the occurrence of earthquakes, for the damaged-but-sustained systems a rapid damage evaluation would be possible using the monitored data and damage detection algorithms. Seismic impact can be measured from structures and integrated into the SIS as part of the heterogeneous data being analyzed. Structural damage severities visualized by computer aided design techniques (CAD), as well as variations in landscapes collected by geographical information systems (GIS), are further illustrated in a visual environment (VE) by virtual reality techniques (VR). The visualized seismic impact simulator providing multi-level heterogeneous data will aid the decision makers on evaluating the entire urban area of interest with sufficient information. Elements of a seismic impact simulator are shown in Fig. 21 and conceptualized in Fig. 22.

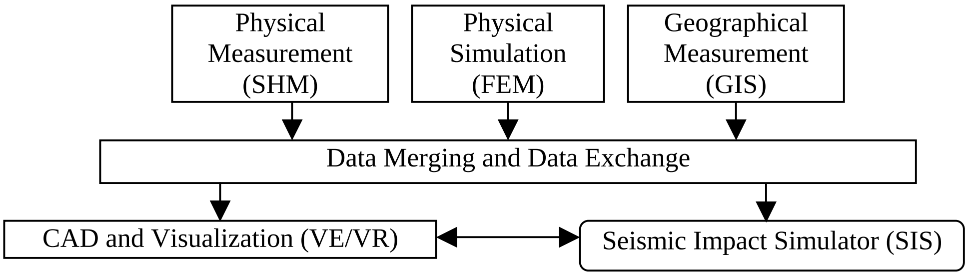


Figure 21. Elements of a Seismic Impact Simulator

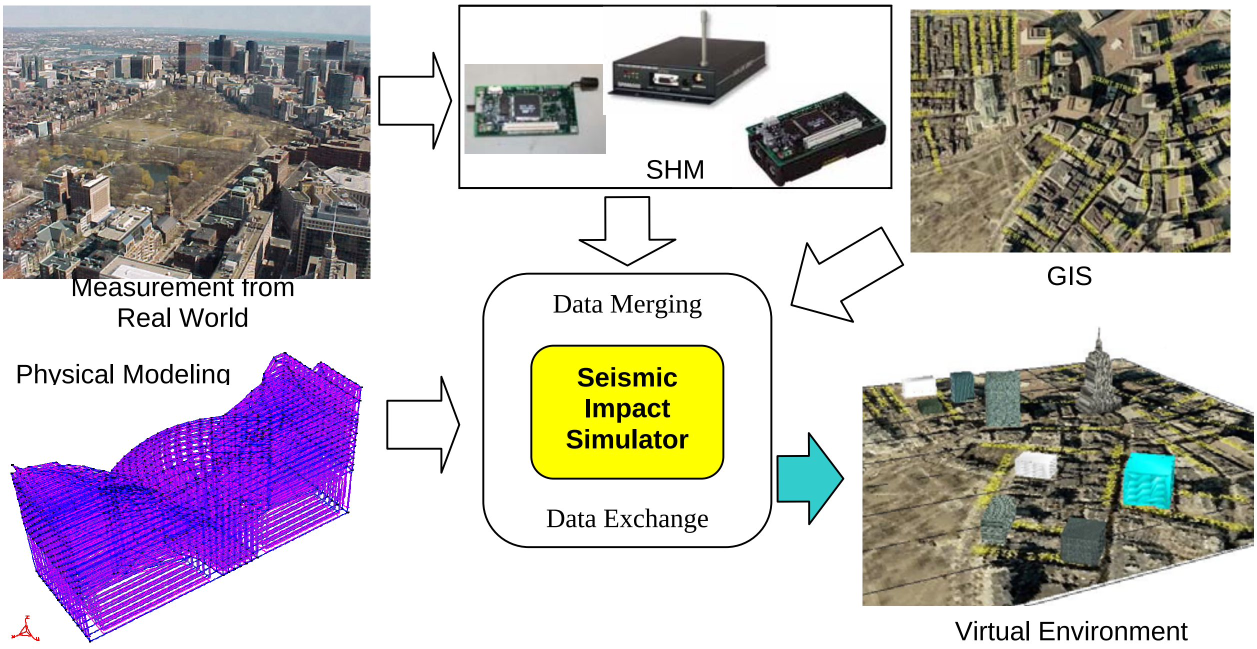


Figure 22. Conceptualization of a Seismic Impact Simulator

CONCLUSION

Components of structural health monitoring (SHM) systems are identified and described. A SHM system design should cover issues of data acquisition, power supply, data transmission, data storage and mining, data processing, and data interpretation. The multidisciplinary feature of SHM is exhibited by its required integrated approach. Among the essential components, data interpretation or health evaluation algorithms plays a critical role in determining the effectiveness of a SHM system. Damage severity is used as the criterion for evaluating structural health condition. Various damage detection algorithms have been developed in the past several decades. Considering the nature of distributed sensor network, modal analysis has been widely applied in theoretical and experimental work. A perturbation analysis of modal-based parameters in terms of structural property changes is provided.
Data interpretation using system identification technique involves the problem of non-uniqueness of solution. A well-defined system model and sufficient amount of measurements in terms of adequate design of sensor networks should be able to improve the situation.
Seismic monitoring system, for the purpose of hazard analysis, local site effects analysis, vulnerability analysis, and estimation of loss, is one application of SHM in earthquake engineering. A prototype, conceptual seismic impact simulator integrating SHM, FEM, GIS, and Virtual Reality (VR) techniques has been proposed as the potential future research direction.
From the current experience of SHM, it is found that the development of SHM is closely linked to the evolving sensing and communication technologies. Micro-Electro-Mechanical Systems (MEMS) technology shrinks the physical size of traditional transducers and decreases the required power level enabling the use of self-powered technology. Wireless communication technology promotes the use of distributed sensor network without increasing installation cost and maintenance effort at an unacceptable rate. It is expected that the emerging technologies from different fields will impact and reshape the design of SHM. However, it is conceivable that some system-level design issues will emerge. For instance, the global design criterion of sensor deployment considering the effectiveness and reliability of the data-collecting system is not clear at present. The use of redundant measurement is still controversial. Without solving

fundamental questions, such as the representation of damage in terms of physical variables, the power of damage detection algorithms will be limited.

The core technique of SHM is the damage detection algorithm. Considering the previous discoveries and experiences, requirements of a robust damage detection/identification algorithm are identified as follows:

- Capability of sensing local disturbances – Incipient damages are always local. To be qualified as an early warning system, damage algorithms should be capable of sensing local disturbances.
- Capability of distinguishing damage events from environmental disturbances and the noise – Collected measurements from the real world are usually contaminated with environmental influences and unidentified noises. To be able to distinguish structural response from the contaminated measurements, efficient de-noising and filtering techniques are required.
- Capability of rapid evaluation – To be able to serve as an emergency warning system and used as a real-time monitoring system, damage detection algorithms need to be computationally efficient to provide the interpretation result in required time. Precision should not be sacrificed by computation speed.

ACKNOWLEDGMENT

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