Structural Health Monitoring of high-rise building structures

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By

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reports on its web site without editorial or peer review.
Abstract

This MQP presents an application of structural health monitoring (SHM) schemes to the large-scale civil engineering structures. The SHM methodology is an integrated model of a nonlinear autoregressive moving average (ARMA) and an eigenvalue analysis framework: (1) the proposed approach first approximate and predict the responses of the structural system under dynamic loads and then (2) eigenvalues from the approximated mathematical model are extracted for structural damage detection using linear system theory. To demonstrate the effectiveness of the proposed approach, 76-story high-rise buildings equipped with a passive tuned mass damper (PTMD) and an active tuned mass damper (ATMD) system under wind loads are investigated, while the uncontrolled original high-rise building structures are used as a baseline model. It is shown from the simulation results that the proposed ARMA-eigenvalue analysis framework is effective in detecting damages from the high-rise building structure under wind loads.
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1. Introduction

Civil structures such as bridges, buildings, dams or wind turbines are complex systems that are vital to the well-being of our society. Safety and durability of civil structures play a very important role to ensure society’s economic and industrial prosperity. Unfortunately, many of our ageing civil structures are deteriorating because of continuous loading, harsh environmental conditions, and, often, inadequate maintenance. For example, in the United States more than 150,000 bridges – about 25% of the U.S. bridges – are considered structurally deficient. In Germany, as another example, more than 80% of the Federal highway bridges show signs of deteriorations, with required repair and maintenance costs estimated at more than €6.8 billion. Generally, all structures deteriorate over time. Therefore, an important task is to reliably assess the structural conditions and to ensure that safety standards are met throughout the operational life of a structure. It is also beneficial for the owners and facility operators of a structure to know the extent of deteriorations and to estimate the remaining life of a structure. The proposed method of achieving this comes in the form of Structural Health Monitoring.

2. Structural Health Monitoring (SHM)

There has been a technological push in developing SHM systems, as a supplement to visual inspections and non-destructive evaluation (NDE), to assess the safety and durability of civil structures. The advantage of visual and NDE techniques is that they give very precise information about the structure; however in the case considered in this paper, of high-rise building structures, they lack the ability to provide uninterrupted measurements about the whole structure. Opposed to a SHM system which is installed permanently on a structure to monitor its conditions on a continuous basis and which gives information about every point in the structure.
SHM systems, in principle, consist of sensors (e.g. accelerometers, temperature sensors, displacement transducers) that are installed in the structure to collect response measurements caused by some external or internal stimuli. The measurements are then transmitted to a centralized computer, which holds the responsibility for storing and processing the data collected from the sensors. Once being stored in the centralized computer, the data can be analyzed automatically by software programs or manually by human experts. Many data analysis approaches have been developed in assessing the integrity of a structure. These approaches can be grouped into two main categories: Local methods and Global methods. First local methods will be discussed, after which global methods will be presented.

2.1. Local methods

2.1.1. Ultrasonic Testing: this method involves a beam of high frequency mechanical vibration or ultrasonic energy transmitted through the test material which is intercepted and reflected by discontinuities back to the receiver. The travel time of the beam is being measured. Knowing the velocity of wave propagation and time, the distance to the discontinuity is estimated. Information about the discontinuity such as location, size and type is displayed on the screen.

2.1.2. Eddy Current Testing (ET): this is a non-destructive testing technique based on inducing electrical currents in the material being inspected and observing the interaction between those currents and the material. Eddy currents are generated by electromagnetic coils in the test probe, and monitored simultaneously by measuring probe electrical impedance.
2.1.3. Magnetic Particle Inspection (MT): this is a method of locating surface and subsurface discontinuities in ferromagnetic materials. It depends on the fact that when the material or part under test is magnetized, magnetic discontinuities that lie in a direction generally transverse to the direction of the magnetic field will cause a leakage field. Therefore the presence of the discontinuity is detected by the use of finely divided ferromagnetic particles applied over the surface, with some of the particles being gathered and held by the leakage field. This magnetically held collection of particles forms an outline of the discontinuity and generally indicates its location, size, shape, and extent.

2.1.4. Liquid Penetrant Testing (PT) and inspection: this is a non-destructive testing method of revealing discontinuities that are open to the surfaces of solid and essentially nonporous materials. Indications of a wide spectrum of flaw sizes can be found regardless of the configuration of the work piece and regardless of flaw orientations.

2.1.5. Visual Inspection: this is also a non-destructive testing technique that provides a means of detecting a variety of surface flaws, such as: corrosion, contamination, surface finish, and surface discontinuities on joints (for example, welds, seals, solder connections, and adhesive bonds). Visual Inspection is the most widely used method for detecting and examining surface cracks, which is particularly important because of their relationship to structural failure mechanisms.

Although most of these local methods have a higher precision than global methods, they are best suited for analyzing small structural elements. When applied to entire civil structures they lack the ability to give the user a timely and efficient analysis of the entire structure. In the monitoring
of high-rise building structures this becomes even more difficult, because the user is faced with a large and not easily accessible structure. Also, continuous monitoring of the structure is hardly possible. If wanted, local methods can be used in correlation with global methods, in areas where damage is identified and a more precise measurement is needed. For the reasons discussed above the use of global methods will be discussed in analyzing high-rise building structures.

2.2. Global Methods

Global methods are focused on providing the user with information about the whole structure. Although not so exact as the local methods, it has the advantages that it provides real-time information and allows the monitoring of the whole structure. Some of the general known global methods are presented in the next section. This is important because a comparison is needed between the vibration-based method, which is also a global method, and all of the other global methods. It will be shown why the vibration-based method is best suited for this example.

2.2.1. Embedded fiber-optics sensors and networks: this method can be very efficient for initial point damage recognition. Any internal micro-displacements of material can be detected with extremely high accuracy by destroyed wavelet or sensor location. This solution has several drawbacks. Firstly, it can be applied only to new structures, by “embedding” at creation stage. For historic buildings or for already existing bridges, tunnels, etc. this method is inapplicable. Another important question in articles in this field is the question regarding the influence of fiber-optical embedded nets on properties of the structural element’s components (e.g. concrete,
reinforcing steel). Finally, it can be noted that this group of materials is not very economical in power consumption.\(^1\)

2.2.2. *Optical inspection methods and optoelectronic scanning*: the methods were developed in 1970–1980s, parallel with surveying techniques. Further achievement was stopped because of natural limitations of hardware capacity and accuracy. Also during this time intensive development of various remote sensors started. As a result, in the last 20 years technical parameters (especially technical reliability and accuracy) of electromechanical systems, as well as laser sources and optical sensing evolved. So, it should be considered that it is reasonable to use the properties of optical systems. Some studies in this branch of SHM\(^2\) offer many interesting ideas. But unfortunately, these systems do not have technically completed solutions\(^3\), or are not convenient for automation in the proposed form.\(^4\)

2.2.3. *Laser scanning systems*: these methods are being used successfully in the field of 3D-object visualization and recognition, which uses unique properties of laser rays for surface highlighting. These properties are: fixed frequency of light emission, and correspondingly response; high ability of scanning element to be focused, spatially compressed; well-known methods of the noise protection of optical channel. The difference between various systems is only in the laser emission power, camera resolution, and the price. However, according to their geometry, such systems need a “reference background base” (a background geometric

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\(^{1}\) (S.F. Masri 1994)
\(^{2}\) (Athavale 1990) (Tyrsa 1973)
\(^{3}\) (Tyrsa 1973)
\(^{4}\) (Athavale 1990)
constant/reference for measuring dimensions). These systems are not acceptable for SHM of high-rise building structures because of the natural difficulties with such “reference background”.

2.2.4. 3D terrestrial laser scanning: this is a relatively new, but already revolutionary, surveying technique. The survey yields a digital data set, which is essentially a dense “point cloud” where each point is represented by a coordinate in 3D space. The most important advantage of the method is that a very high point density can be achieved, in the order of 5mm to 10mm resolution. But this technique is designed for large area observation from a relatively long distance. Because of the reconstruction, algorithms have a probabilistic character, which increase the error. Moreover, for this kind of a task, 3D terrestrial laser scanners use very expensive units, like a superpower laser source, supersensitive receivers, etc. So, in conclusion it can be said that terrestrial laser scanners are a good tool for surveying tasks, however not so much for SHM of high-rise building structures.

2.2.5. GPS: this is also another method in SHM; however, the GPS technology does not yet have the degree of accuracy needed for SHM of high-rise building structures. Also, because the gathering and processing of the data cannot be done in real time, the user gets a delayed warning of structural failure. All of these properties coupled with the fact that currently it can be an expensive technic to use, makes GPS not suited for the case presented in this paper.

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5 (Winkelbach 2006)
6 (Slob 2004) (Baltsavias 1999)
7 (Baltsavias 1999)
8 (A.Knecht 2001)
2.2.6. Conclusion: The drawbacks of these methods force us to look further for a global method of applying SHM to high-rise building structures. The main advantage of the vibration-based method is that the sensors are less expensive, more durable and easier to install than the other methods. Also, the fact that it can give all the information that is needed in real-time without delay and with sufficient accuracy makes this the best method for high-rise building structures.

3. Vibration-based damage detection

A comparatively recent development in the health monitoring of civil engineering structures is vibration-based damage detection. Vibration characteristics of a structure are directly affected by the physical characteristics of the structure. As the structure is damaged, the stiffness of the structure is changed and so the vibration characteristics change as well. Therefore, measurement and monitoring of vibration characteristics should theoretically permit the detection of both the location and severity of damage.

This method offers several advantages. One of which is that the location of damage need not be known beforehand. Another is that often the sensors required to measure the vibration characteristics need not be located in the vicinity of the damage. In addition, a limited number of sensors can, at times, provide sufficient information to locate the damage and assess its severity, even in a large and complex structure. Lastly, vibration measurements do not require use of bulky equipment, except when forced vibration tests are carried out.

An important part of this method is processing the data obtained from the sensors. Different methods have been developed to approximate and predict the response of the structure under excitation forces. To extract damage sensitive features from the data series the ARMA
model (Auto-Regressive Moving Average) was used in this example. The main reason for using it is that ARMA was previously used with great success by many authors.\(^9\)

### 3.1. Auto-Regressive Moving Average (ARMA)

The auto-regressive moving average (ARMA) time series model is one possibility of using output data based system identification methods. Time series analysis methods using the ARMA model were employed by various researchers with great success\(^10\). This method offers the distinct advantage of requiring only data from the undamaged structure, not necessarily needing the input data (e.g. wind force). Once a specific model design is established through the time series analysis method, damage can be identified. Damage sensitive features are obtained by observing changes in the coefficients\(^11\) of the AR time series.

Nair et al.\(^12\) have used the ARMA and modeled a new damage-sensitive feature (DSF) which is a function of the first three AR components. The results provide effective representation of damage and localization. Various applications in time series analysis usually assume linear relationship among the past values of predicted variables\(^13\). Although the linear assumptions make it easier to analyze the mathematical manipulation of the models, it can also lead to inadequate representations when non-linear behavior of structures is considered.

In order to reduce this inaccuracy, nonlinear based ARMA model was developed. By taking out the assumption from the previous models nonlinear based, ARMA model allows it to be used on complex and large structures (i.e. actively controlled high-rise building structures). Although it

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\(^9\) (A. S. K. Krishnan Nair 2006), (P.C. Chang 2003), (P. Omenzetter 2006)


\(^12\) (A. S. K. Krishnan Nair 2006)

may be challenging to develop an appropriate mathematical model, it produces more accurate results.

3.2. Nonlinear ARMA (NARMA)

NARMA time series model is proposed to develop the data based model. Prediction error is minimized by using the least square method which is based on a least squares minimization criterion. The first three AR coefficients which are related to poles of the structural system are used to form a damage sensitive feature. The following equation represents the ARMA model:

$$y(n) = \sum_{i=1}^{P} a(i)y(n - i) + \sum_{j=0}^{Q} b(j)x(n - j) + e(n)$$

where $P$ and $Q$ represent the optimal AR and MA model orders, respectively. The term $e(n)$ is considered a noise source or prediction-error term. The parameters $a(i)$ and $b(j)$ represent to-be-estimated coefficients of the AR and MA terms, respectively. The candidate vectors are the following: $y(n-1), ..., y(n-P)$ and $x(n), ..., x(n-Q)$. These candidate vectors can be arranged as the matrix shown below:

$$
\begin{bmatrix}
  y(0) & x(1) & y(-1) & x(0) & \cdots & y(1-P) & x(1-Q) \\
  y(1) & x(2) & y(0) & x(1) & \cdots & y(2-P) & x(2-Q) \\
  \vdots & \vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
  y(n-1) & x(n) & y(n-2) & x(n-1) & \cdots & y(n-P) & x(n-Q) \\
  \vdots & \vdots & \vdots & \vdots & \ddots & \vdots & \vdots \\
  y(N-1) & x(N) & y(N-2) & x(N-1) & \cdots & y(N-P) & x(N-Q)
\end{bmatrix}
$$

where $N$ is the total number of data points. In a nonlinear ARMA model, the matrix can be expanded not only to include projects between the input and output itself, but the cross products between the input and output terms as well. With the new candidates of linearly independent vectors, least-square analysis performed

$$y(n) = \Theta_0 x(n) + e(n)$$
where $\emptyset = [w_0, w_1, w_2, ..., w_R]$ and $R$ is the number of selected linearly independent vectors.

$$\theta_g = [g_0, g_1, g_2, ..., g_R]^T$$  \hspace{1cm} (4)

$g_i$ is the coefficient estimate of the ARMA model. The objective is to minimize the equation error, $e(n)$, in the least-squares sense using the criterion function defined as follows:

$$J_N(\theta_g) = [y(n) - \theta_g^T\emptyset]^2$$  \hspace{1cm} (5)

The criterion function in (5) is quadratic in $\theta_g$, and can be minimized analytically with respect to $\theta_g$, yielding the following equation:

$$\hat{\theta}_g = [\emptyset\emptyset^T]^{-1}\emptyset y(n)$$  \hspace{1cm} (6)

With the obtained coefficients, calculate every $\bar{g}_m^2\bar{w}_m^2$, and rearrange the $w_m$ in descending order. Note that the over-bar represents the time average. At this step of the algorithm, the number of candidate vectors, $w_m$, necessary for obtaining proper accuracy needs to be chosen. This approach is taken in order to retain only the $w_m$ that reduce the error value significantly. If either negligible decrease or increase in the error value by adding an additional $w_m$ is found, then those $w_m$ are dropped from the model. Once only those $w_m$ that reduce the error value significantly are obtained, the nonlinear ARMA model terms are estimated using the least squares method.

Damage in the structure can be assessed through the change in the first three parameters if the ARMA, the process is explained in the following section. The ARMA model can be applied to the $z$-transform function. As the ARMA model is applied to the transfer function, the poles effectively define the system response because the transfer function completely represents a system characteristic equation. Applying the $z$-transform to the given ARMA equation and ignoring the effect of the error term, the equation is given by

$$Y(n) = \sum_{i=1}^{p} a(i)z^{-i}Y(z) + \sum_{j=0}^{Q} b(j)z^{-j}X(z)$$  \hspace{1cm} (7)
where \( Y(z) \) and \( X(z) \) are the \( z \)-transforms of the \( y(n) \) and \( x(n) \), respectively. The transfer function of a stabilized minimum ARMA process has the form in the \( z \) domain \( H(z) \) is derived as:

\[
H(z) = \frac{Y(z)}{X(z)} = \frac{b_0 + b_1 z^{-1} + b_2 z^{-2} + \cdots + b_Q z^{-Q}}{1 - a_1 z^{-1} - a_2 z^{-2} - \cdots - a_P z^{-P}}
\]  

(8)

The denominator of the transfer function \( H(z) \) is a characteristic equation of order \( p \). Since the denominator has real coefficients, the root of the polynomial equation can be defined to be the system poles.

\[
[z^p - a_1 z^{-1} - a_2 z^{-2} - \cdots - a_p] = 0
\]

(9)

Using the theory of the polynomial roots, the first three coefficients can be as follows:

\[
\sum_i z_{\text{pole}, i} = a_1
\]

(10)

\[
\sum_{i,j} z_{\text{pole}, i} z_{\text{pole}, j} = -a_2
\]

(11)

\[
\sum_{i,j,k} z_{\text{pole}, i} z_{\text{pole}, j} z_{\text{pole}, k} = a_3
\]

(12)

By applying the Laplace transformation equation to the poles of the transfer function, the modal natural frequencies and the critical damping ratio can be calculated.

\[
z_{\text{pole}} = e^{-\xi w_n \Delta t \pm j \sqrt{1 - \xi^2 w_n^2 \Delta t}}
\]

(13)

Any damage to a structure will cause changes in the system’s stiffness and damping. These changes can be quantitatively measured by the migration patterns of the transfer function poles.
4. Simulation model

In order to test this model on the chosen structure, some structural details must be given first. Also the excitation, which in this case is wind, must be discussed. These notions are explained in the next two sections, after which application of the ARMA model follows. Damage detection and results are explained last.

4.1. Structure

The building considered is a 76-story 306 m office tower proposed for the city of Melbourne, Australia as shown in Figure 2 and Figure 3. This is a reinforced concrete building consisting of a concrete core and concrete frame. The core was designed to resist the majority of wind loads whereas the frame was designed to primarily carry the gravitational loads and part of the wind loads. All relevant structural analyses and design had been completed; however, it was not built due to an economic recession. The building has a square cross section with chamfer at two corners as shown in Figure 3.

Figure 1 Structure
The total mass of the building, including heavy machinery in the plant rooms, is 153,000 metric tons. The total volume of the building is 510,000 m³, resulting in a mass density of 300 kg per cubic meter, which is typical of concrete structures. The building is slender with a height-to-width ratio (aspect ratio) of $306.1/42=7.3$; therefore, it is wind sensitive. The perimeter dimension for the center reinforced concrete core is 21 by 21 m. The reinforced concrete perimeter frame consists of columns spaced 6.5 m apart, which are connected to a 900 mm deep and 400 mm wide spandrel beam on each floor. There are 24 perimeter columns on each level with six columns on each side of the building. The lightweight floor construction uses steel beams with a metal deck and a 120 mm slab. The compressive strength of concrete is 60 MPa and the modulus of elasticity is 40 GPa. Column sizes, core wall thickness, and floor mass vary along the height, and the building has six plant rooms. A passive tuned mass damper (PTMD) with an inertial mass of 500 t is installed on the top floor. This is about 45% of the top floor mass, which is 0.327% of the total mass of the building. Besides the case of the structure with the PTMD installed, the
structure with an Active Tuned Mass Damper (ATMD) installed, as well as the structure without any control system installed will also be analyzed. These systems will be discussed in the next section.

4.2. Passive Tuned Mass Damper and Active Tuned Mass Damper

A tuned mass damper, also known as a passive tuned mass damper (PTMD) or harmonic absorber, is a device mounted in structures to reduce the amplitude of mechanical vibrations. Their application can prevent discomfort, damage, or outright structural failure. They are frequently used in power transmission, automobiles, and buildings.

A great deal of interest among civil engineers has been focused upon the control of high-rise building structures under different dynamic forces such as earthquakes and wind loads. For the last two to three decades, control devices have seen much development especially in the case of using a mass to absorb energy due to different excitations. Some examples of buildings having such devices installed are Citicorp Center in New York and the John Hancock Tower in Boston with a 400-ton and 300-ton TMD installed, respectively.\(^{14}\)

Looking for a way to improve the effectiveness of the TMD, the introduction of a controlling force was considered. Morison and Karnopp theoretically investigated such an active vibration control approach as compared with a conventional passive TMD\(^ {15}\). Lund studied the feasibility of the method by using an actuator and a pneumatic spring\(^ {16}\). Chang and Soong\(^ {17}\)

\(^{14}\) (Isyumov. N. 1975)
\(^{15}\) (D 1973)
\(^{16}\) (A. 1979)
\(^{17}\) (Chang 1980)
further investigated this idea as well as other authors\textsuperscript{18} who tried to optimize the design of such a system.

While the explanation as to how these devices work is not the focus of the MQP, conceptual drawings of these devices are provided below.

\textsuperscript{18} (Isao Nishimura 1992)
The dynamic force that is considered in all three cases is wind. In order to apply the ARMA model, some concepts about the wind force, like vorticity\textsuperscript{19}, as well as how this force was measured in the wind tunnel must be presented. All of these will be discussed in the next section.

4.3. Wind Excitation

Wind force data acting on the benchmark building in along-wind and across-wind directions were determined from wind tunnel tests. A rigid model of the 76-story benchmark building was constructed and tested in the boundary layer wind tunnel facility at the Department of Civil Engineering, the

\textsuperscript{19} Vorticity is the tendency for elements of the fluid to "spin."
University of Sydney, Australia. The wind tunnel is of the open circuit type with a working section of 2.4 m $32.0$ m and a working length of 20 m. An appropriate model of the natural wind over a suburban terrain was established using the augmented growth method, which included a combination of vorticity generators spanning the start of the working section and roughness blocks laid over a 12 m fetch length of the working section.\textsuperscript{20}

Knowing the structure and the excitation (i.e. wind) the data that was recorded and used further on in applying the ARMA model and identifying damage can be presented.

4.4. Measurements

The following plots represent the data that used in the examples analyzed. These represent the recorded acceleration by sensors in the structure. For each of the three cases, the data from the Undamaged structure was plotted first, followed by the data from the Damaged structure and finally the two previous data overlapped were plotted in order to get a sense of the damaged/undamaged status of the structure.

\textsuperscript{20} (Jann N. Yang April 2004)
Figure 9 Data from the damaged and undamaged structure with the ATMD installed

Figure 10 Data from the damaged and undamaged structure with the PTMD installed
In the next section the parameters extracted from the ARMA model will be presented for each case analyzed. Also a classification algorithm will be discussed, and the conclusion that the ARMA model was successful in implementing SHM to high-rise building structures will be stated. As well as the fact that the ATMD device is better suited in this situation as the PTMD.

4.5. AR parameters

The relationship between AR parameters and modal parameters, explained in section 3 of this paper, makes understandable the use of the AR parameters in analyzing the changes in stiffness, mass or damping properties of the structure.\textsuperscript{21} This is the reason why the AR parameters were used to identify damage in the three case studies considered.

\textsuperscript{21} (E. Peter Carden, ARMA modelled time-series classification for structural healthmonitoring of civil infrastructure February 2008)
The developing of a classification algorithm, in this case the damage sensitive feature (DSF) was made using the data acquired from the structure with the Passive Tuned Mass Damper (PTMD) installed. After finding a classification algorithm that was able to clearly differentiate the damaged structure from the undamaged structure, it was applied in the other two examples (i.e. the uncontrolled structure and the structure with the ATMD installed) to enforce the reliability of the newly developed algorithm.

4.6. Developing the formula for the damage sensitive feature (DSF)

The first step in developing the DSF was to extract the AR parameters. This was done by sampling the data and applying the ARMA model. After which the AR parameters were extracted for every sample in the two data series (damaged and undamaged which are tabulated below):

<table>
<thead>
<tr>
<th>Undamaged structure with PTMD</th>
<th>α₁</th>
<th>α₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>0.99965284307110700000000</td>
<td>0.0000005594921084332440</td>
</tr>
<tr>
<td>Sample 2</td>
<td>0.99962267973900900000000</td>
<td>0.0000000912624174205285</td>
</tr>
<tr>
<td>Sample 3</td>
<td>0.99960787837969800000000</td>
<td>0.0000001443665453677080</td>
</tr>
<tr>
<td>Sample 4</td>
<td>0.99976359001560800000000</td>
<td>-0.0000000500379205869373</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Damaged structure with PTMD</th>
<th>α₁</th>
<th>α₂</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>0.99971360079010400000000</td>
<td>0.00000004036431283991150</td>
</tr>
<tr>
<td>Sample 2</td>
<td>0.99969434927027600000000</td>
<td>0.00000004512160164961130</td>
</tr>
<tr>
<td>Sample 3</td>
<td>0.99974357120081000000000</td>
<td>0.0000002037830472131930</td>
</tr>
<tr>
<td>Sample 4</td>
<td>0.99890097811431800000000</td>
<td>0.000000331830923858400</td>
</tr>
</tbody>
</table>

Table 1 AR parameters from the data of the structure with the PTMD
The ARMA model used only two terms to approximate the data; as a consequence only two parameters ($\alpha_1$ and $\alpha_2$) were extracted. The selection of only two terms in the ARMA model was sufficient as shown in Figure 12 where the approximation of the data set by the ARMA model is presented. It can be seen from the zoomed in window (Figure 13), that the two data sets represented in blue and red, are accurately approximated by the ARMA model shown with a discontinues grey line. Attention is drawn to the fact that it was necessary to zoom in quite a lot in order to see the approximation done by the ARMA model.
Figure 12 Approximation of damaged and undamaged data

Figure 13 Zoomed in approximation of damaged and undamaged data
To identify if damage was present in the building when the data was collected, a series of classification algorithms were applied and eventually the one which was the best fit for the given structure was used in all of the three examples. The first one tried was, the classification algorithm developed by Kincho H. Law\textsuperscript{22}:

\[
DSF = \frac{\alpha_1^2}{\sqrt{\alpha_1^2 + \alpha_2^2 + \alpha_3^2}}
\]  \hspace{1cm} (14)

However, because only two parameters were obtained, the classification algorithm was modified as follows:

\[
DSF1 = \frac{\alpha_1^3}{\sqrt{\alpha_1^2 + \alpha_2^2}}
\]  \hspace{1cm} (15)

where $\alpha_1$ and $\alpha_2$ are the first two AR parameters and DSF stands for damage-sensitive feature. The DSF values were plotted for each sample made in the damage and undamaged data and the following plot was obtained:

\hspace{1cm}

\textsuperscript{22} (A. S. K. Krishnan Nair 2006)
As seen in the figure above, there is no discernible separation between the “damage” and “no damage” plot points. Seeing that a difference between the undamaged and damaged data could not be observed, the DSF formula was modified by normalizing the second parameter. This was done because the second parameter changed much more than the first one, but being approximately 8 orders smaller than the first parameter (please see Table 1), this difference could not be detected. The modified formula looks as follows:

$$\text{DSF2} = \frac{a_2^2}{\sqrt{a_1^2 + a_2^2}}$$

(16)

This resulted in the following plot:
Because again the plot didn’t show a clear difference between the damaged and the undamaged data and not wanting to modify the formula again, the sample size was increased$^{23}$. Keeping the modified DSF equation the following plot was obtained:

$^{23}$ From a sample size of 20,000 to one of 200,000 out of a data set of 900,001 points (acceleration vs. time)
This is a better representation of the damaged and undamaged status of the high-rise building structure, because a difference can be seen between the DSF points from the two data sets. Although a difference can be observed between the “no damage” points and their corresponding “damage” points, it can be seen that for sample no. 1 the “no damage” is above the “damage” point, but for sample no.2 this is the other way around. Because of this variation a more accurate representation was needed, so going back to the first DSF formula used and dividing the first parameter by $10^8$, this being the difference in order size between the first and the second parameter (please see Table 1), the following formula resulted:

$$DSF3 = \frac{(\alpha_1 \times 10^{-8})^2}{\sqrt{(\alpha_1 \times 10^{-8})^2 + \alpha_2^2}}$$ (17)
This increased the effect that the second parameter has on the DSF, thus making the difference between the damage and the undamaged status more obvious. The following plot resulted:

![Figure 17 First parameter normalization with modified order number of structure with PTMD](image)

This shows a very clear difference between the damaged and undamaged data, as all of the “no damage” points are above their corresponding “damage” points. Because this is the best result so far, Eq.17 (i.e. DSF3) was used in the classification of the other two examples.

Next the building without any control device was analyzed. Although in most cases the damage presented in the following case would be bigger than in the previous case, where the structure had a PTMD\textsuperscript{24} installed, this is not always the case. The reasons vary from the PTMD

\textsuperscript{24} Passive Tuned Mass Damper
not being designed correctly in the first place, to the fact that the stiffness of the building changed from when the PTMD was designed and installed.

5. Results

5.1. Uncontrolled structure

The following plot shows the approximation of the data from the uncontrolled structure (acceleration vs. time) using the ARMA model:

![Figure 18 ARMA model for uncontrolled structure](image)

As it can be seen there is damage present, which was expected. This can be said, seeing that the two data sets do not overlap perfectly when plotted one on top of the other. The parameters extracted from the ARMA model are summarized in the next table:
Table 2 AR parameters from the data of the uncontrolled structure

<table>
<thead>
<tr>
<th>Sample</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>1.0000007826740400000000</td>
<td>0.0000002261484449390570</td>
</tr>
<tr>
<td>Sample 2</td>
<td>1.0000009999412900000000</td>
<td>0.0000003098486129142430</td>
</tr>
<tr>
<td>Sample 3</td>
<td>0.9999990927942970000000</td>
<td>0.000000297914684328140</td>
</tr>
<tr>
<td>Sample 4</td>
<td>0.9999990338798790000000</td>
<td>0.0000002378781636242920</td>
</tr>
</tbody>
</table>

The plot of the DSF3 values, which were calculated using Eq.17, is shown below:

Figure 19 DSF3 values for the uncontrolled structure
5.2. Structure with passive tuned mass damper (PTMD)

For comparison purposes the results for the PTMD are presented, again, below. First the approximation of the data set by the ARMA model and then the DSF3 values plotted for every sample:

![Figure 20 ARMA model for structure with PTMD](image_url)
As it can be seen from Figure 19 the level of damage in the uncontrolled structure is far less than in the structure controlled with the help of the PTDM. As stated before, this doesn’t happen very often, but in this case it can be seen how the PTDM, although very good for some situations, didn’t work and actually made matters worse in this case. The PTDM’s positive effects in other situations shouldn’t be neglected, just because for this situation it did not work as expected.

The final case is the Active Tuned Mass Damper (ATMD). This is different from the PTDM as it molds to the structures needs. This means that even though the stiffness of the structure changes or the excitation is different than that considered when the design of the structure was made, the ATMD makes up for these situations by calculating what the displacement of the mass should be and applying that displacement exactly at the time needed.
5.3. Structure with ATMD installed

As with the previous two examples, the ARMA approximation is presented first, in the next graph:

![ARMA approximation of the structure with the ATMD installed](image)

It can be seen that the two data sets overlap better than in the previous two examples. This shows that less damage is present in this case than in the other two cases. In the following table the AR parameters extracted for this data set are summarized:
<table>
<thead>
<tr>
<th>Sample</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>0.99912251961295100000000</td>
<td>0.00000006683245778848420</td>
</tr>
<tr>
<td>Sample 2</td>
<td>0.99919896429187500000000</td>
<td>0.0000000088746086811903</td>
</tr>
<tr>
<td>Sample 3</td>
<td>0.99906934987294700000000</td>
<td>0.0000000098497870995381</td>
</tr>
<tr>
<td>Sample 4</td>
<td>0.9991910301522360000000</td>
<td>0.0000000661440210270587</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample</th>
<th>$\alpha_1$</th>
<th>$\alpha_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>0.9993931355567010000000</td>
<td>0.00000005456324599369890</td>
</tr>
<tr>
<td>Sample 2</td>
<td>0.9993091937881470000000</td>
<td>0.00000003274016121759270</td>
</tr>
<tr>
<td>Sample 3</td>
<td>0.9994237051482420000000</td>
<td>-0.0000000322158514919161</td>
</tr>
<tr>
<td>Sample 4</td>
<td>0.9979438409874510000000</td>
<td>0.00000003620475843197190</td>
</tr>
</tbody>
</table>

Table 3 AR parameters from the data of the structure with the ATMD installed

Applying Eq.17 (i.e. DSF3) to calculate the DSF values and plotting these for every sample that was investigated, the damage level in the structure was assessed.
5.4. Discussion

To get a better comparison the graphs from all three cases will be presented below:
The first graph represents the DSF3 values from the uncontrolled structure, the second graph represents the structure with the PTMD installed and the third graph represents the structure with ATMD.

Figure 25 DSF3 values for the structure with PTMD

Figure 26 DSF3 values for the structure with ATMD

The first graph represents the DSF3 values from the uncontrolled structure, the second graph represents the structure with the PTMD installed and the third graph represents the structure with ATMD.
the ATMD installed. We would expect to see the difference from the damaged and the undamaged data to decrease from the first graph to the third graph, but this is not the case. For this reason the sample size was decreased. As a result increasing the number of samples and getting more data points on the graph. The graphs are presented next:

![Graph showing DSF3 values for the uncontrolled structure (small sample size)](image)

**Figure 27** DSF3 values for the uncontrolled structure (small sample size)
Figure 28 DSF3 values for the structure with PTMD (small sample size)

Figure 29 DSF3 values for the structure with ATMD (small sample size)
As it can be seen the difference between the damaged and the undamaged data is getting smaller as better control systems are applied to the structure. This can be said because the “damage” points get much closer to the “no damage” points, telling that there is not much difference of the damaged structure from the undamaged structure which is the baseline model. As a result it is safe to say that the structure’s damage level decreases.

So far only the structure as a whole was analyzed, but a more localized analysis is also possible. For example the damage level on different floors of the structure can be assessed. In the next section, data from different floors in the three cases of the structure (i.e. uncontrolled, controlled with PTMD and controlled with ATMD) will be analyzed.

5.5. Localized analysis

The plotted DSF3 values (calculated using Eq.17) for the following floors are presented: 50, 55, 60, 65, 70 and 75. For every floor the DSF3 values are analyzed for each of the three cases of the structure considered: the uncontrolled structure, the structure with the PTMD installed and with the structure with the ATMD installed.
Figure 30 DSF3 plotted for the 50th story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
55\textsuperscript{th} story:

Figure 31 DSF3 plotted for the 55\textsuperscript{th} story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
Figure 32 DSF3 plotted for the 60th story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
Figure 33 DSF3 plotted for the 65\textsuperscript{th} story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
70\textsuperscript{th} story:

Figure 34 DSF3 plotted for the 70\textsuperscript{th} story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
Figure 35 DSF3 plotted for the 75th story of the: a) uncontrolled structure, b) structure with PTMD, c) structure with ATMD
As it can be seen from the graphs presented above, the method chosen of detecting the damage level on each particular floor level was successful. This can be said because a very clear differentiation between the “damage” and “no damage” DSF3 values for every data set can be observed. As well as seeing a bigger difference in the “uncontrolled” structure than in the structure controlled with an ATMD device.

With this last example it can be concluded that the developed method of applying SHM to a high-rise building structure was successful. The method makes use of the vibration-based method coupled with the ARMA model, for approximating and predicting the data, and the newly developed classification algorithm in the form of the DSF3 formula.

6. Conclusion

The focus of this MQP was to search for the best method of applying Structural Health Monitoring (SHM) to actively/passively controlled high-rise building structures under ambient wind loads. After investigating a variety of structural health monitoring methods, the vibration-based Method was chosen. This method was used coupled with the AutoRegressive Moving Average (ARMA) model of approximating and predicting the data sets. Also a classification algorithm in the form of a formula for the damage sensitive feature (DSF) was developed.

This methodology was applied to a structure in three cases: uncontrolled, with a passive tuned mass damper (PTMD) installed and with an active tuned mass damper (ATMD) installed. Also the structure was analyzed globally (i.e. one data set for the entire structure) as well as locally (i.e. individual data sets for five stories). The findings are as follows:

- The method chosen of applying SHM to high-rise building structures was successful in assessing the damage level both globally as well as locally;
• The best controlling device was found to be the ATMD in this situation;
• The sample size played a very important role in our results and for future applications, this subject must be treated with great importance;
• The damage sensitive feature (DSF) developed was greatly influenced by the relation among the AR parameters;
• The vibration-based method coupled with the ARMA model and the classification algorithm developed is the most efficient (i.e. cost and time) and accurate way of applying SHM to high-rise building structures.

7. Reference
A. Csipkes, S. Ferguson, T.W. Graver, T.C. Haber, A. Mendez, J. W. Mille. The maturing of optical sensing technology for commercial applications. Atlanta, Georgia 30345: Micron Optics Inc., 1852 Century Place, , n.d.


