Application of Smart Façade System in Reduction of Structural Response During Wind Loads

by

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Certificate of authorship/originality

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ABSTRACT

Strong wind causes severe shaking, mostly lateral, over a large area that applies strong excitation to building structures. These winds are extreme actions, from which buildings may not survive unless being properly designed in advance. In recent years, many new devices, such as energy-absorber systems, have been introduced. But, most of them have some disadvantages such as complexity of design and requirement of large spaces for installation. To date the engineering community has seen structural facade systems as non-structural elements with a high aesthetic value and a barrier between the outdoor and indoor environments. As an integral part of all structural buildings, they are susceptible to potential failure when subjected to severe environmental forces such as earthquake and high wind in case they are not designed properly. Wind loads can potentially impose a significant loading on the facade system and may lead to damage and racking in the case of an insufficient connection detailing correspondingly.

The role of facades in energy use in a building has been also recognized and the industry is witnessing the emergence of many energy efficient facade systems. Despite these advancements, the facade has been rarely considered or designed as a potential wind-induced vibration absorber for structural buildings. Development and implementation of advanced facade systems for enhancing the wind response of building structures have been a topic of debate for structural and architectural engineers. Considering this issue,

An alternative method using façade systems incorporated with energy-absorbing devices were proposed in order to damp the amount of energy transferred to the main structure during wind activities.

Various configuration and specification scenarios of the proposed system were suggested in this thesis. Multiple design variations were evaluated as well. To prove the concept and find the optimum value of damper properties, a series of non-linear structural analysis and finite element modelling was done in ANSYS program. First, conventional façade brackets were replaced with the so-called smart elements, which can have back and forth movement during wind load. Predefined elastic-plastic behaviour was suggested for the façade bracket elements in a double skin façade system. Second, façade bracket properties in terms of stiffness and damping of the proposed system were optimized to obtain the desired response. Third, the potential of utilizing a movable exterior facade in a double-skin facade was investigated and it was found that, with optimal choices of façade out-plane movement and appropriate bracket

stiffness, a substantial portion of wind-induced vibration energy can be dissipated, which could lead to avoiding expensive wind designs.

A series of dynamic time history analyses was also carried out to determine the behaviour and response of the proposed system on typical concrete frame structures under different intensity wind. ANSYS and Matlab programs were used for the numerical analyses in all phases of the feasibility study. The initial works demonstrated that the wind response for mid- and high-rise structural buildings subjected to wind loads can be substantially reduced by the introduction of a smart design of a double skin system. Application of flexible connections in façade systems can, if properly designed, reduce the top acceleration response of structural models in comparison with the case without flexible connections.

KEYWORDS: Façade Systems, Multi-Skin Façade, Tall Building, wind Load

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Chapter 1

1 INTRODUCTION

1.1 The Need for Using Façade as a Structural Element

The development and increasing use of light-weight and high-strength materials in construction of tall buildings, offering greater flexibility and reduced damping, has increased tall building susceptibility to dynamic wind load effects. The main associated risk is resonant oscillations induced by von-Karman like vortex shedding at or near the natural frequency of the structure caused by flow separation. The effects of dynamic wind loading increase proportionally with the power of the wind, causing tall buildings to pay a significant material price to increase the natural frequency and/or provide damping. In particular, crosswind response often governs both the strength and serviceability (human comfort) design criteria. While both tuned mass damper (TMD) or tuned liquid dampers (TLD) have merit, these types of dampers often come at the expense of valuable leasable area and high construction costs, due to increased structural requirements for mass and stiffness.

To date, very few engineers and architects have exploited the potential of façade systems as an energy absorbing system to combat wind loads. Most attempts so far have considered façade as an add-on with no or little structural contribution and this is evidenced by the exclusion of façade systems in computer modelling of tall buildings as an analysis tool. Double-skin façade systems (DFS) are becoming very popular for improving the sustainability of commercial buildings in Australia and overseas.

1.2 Thesis Aims, Objectives and Methodology

The effects of high winds on high rise and mid-rise buildings are well known by the engineering community. In the case of wind loads, high-rise buildings may be susceptible to excessive deflections and noticeable accelerations and such problems are usually reduced by the adoption of external damper systems. The loss of valuable and prime space coupled with the initial cost of installing large size damper systems has been accepted by building owners with reluctance and any viable alternative system to dissipate wind excitations would be highly appreciated by them. Similarly, for medium size buildings the reliance on damper systems to alleviate wind induced actions has also received much attention in recent years. In these cases, it is proposed to use a moveable façade skin attached to passive devices which are in turn attached to a fixed frame. The energy imparted by wind force can be dissipated by the absorbing façade system with major economic benefits.

1.2.1 Thesis Aims

A new facade damper system will concentrate on different types of flexible/energy absorbing façade systems and their behaviour under wind loading. A substantial number of numerical, analytical and experimental analyses related to the simulated wind performance of facade systems will be performed. The façade system will be an energy absorbing one incorporating specially designed dampers (passive or semi active). Design of a new energy absorbing façade system, fully tested against wind loads, is the ultimate goal of the research.

1.2.2 Objectives

The overall objectives of this research are, therefore, to:

(1) In the first stage, to show the potential and feasibility of the movable facade concept, ANSYS Finite Element software was used to model a 3D frame structure with and without moving façade systems. Various combinations of façade systems in terms of façade mass, façade panel heights, damper locations and number of dampers were analysed.

(2) To determine the optimum values for mass ratio, frequency ratio and damping ratio of façade and its connections. The second and more detailed phase is devoted to finite element analyses. ANSYS Finite Element software is used to model a typical façade unit with moving outer skin. In the analyses, the sensitivity of parameters such as façade panel height, damper locations and number of dampers as well as supporting stiffness of bracket elements in achieving an optimal system is determined.

(3) To prove the feasibility and effectiveness of the new smart façade system, prototype testing is required as part of experimental investigations. As part of the future works of this program, and based on the results of previous works, Permasteelisa Group (the project industry partner) will fabricate and deliver at least one façade unit with designated damper systems (passive or semi-active) for testing.

(4) To conduct a financial viability and cost benefits of the proposed system in terms of initial investment as well as life cycle costs of utilising facade dampers instead of mechanical dampers.

1.2.3 Methodology

Mainly, the methods which will be used to carry out investigations on the feasibility and performance of the proposed system are based on analytical observations and findings. The main part of this research is conducted using computer simulations involving time history nonlinear analyses of simplified models and Finite element analysis for detailed modelling. Material properties (compressive strength of beams and columns) and dynamic properties (stiffness, mass and natural frequency of vibration) of the main structure, damping properties, configuration and location of dampers, facade mass and selected winds are influential parameters.

Based on the above parameters, properties of the optimal façade connection system will be evaluated. At the end, the results of the investigation will be presented in design tables and guidelines which would prove very useful and crucial for the industrial partner in particular, and the façade industry in general.

1.2.3.1 Analytical Façade Models

A series of dynamic time history analyses is carried out to determine the behaviour and response of the proposed system in typical concrete frame structures under different wind force scenarios. To prove the concept and find the optimum value of damper properties, a series of finite element modellings are calculated using the ANSYS program. This research focuses on along-wind response reductions caused by positive pressures on the windward face of the building and negative pressures on leeward face. The reduction of vortex-induced crosswind response is beyond the scope of this research and forms part of a future study.

1.3 Thesis Overview

The introduction outlining the needs for the smart façade system and the thesis aims, objectives and methodology is outlined in Chapter 1.

Chapter 2 provides an overview into different facade systems, general curtain wall façade anatomy and how these parameters affect the behaviour and required performance of façades.

Chapter 3 covers characteristics of wind loads and methods of mitigating wind effects. The characteristics of fluctuating wind forces and the dynamic characteristics of the building are shown in this chapter. Different damper systems are presented based on their effect on the dynamic behaviour of the structures.

Chapter 4 introduces the Finite element models. The author managed the development of properties of façade wind load analysis software; Ansys, that utilises the Finite element model presented in this thesis. Comparative calculations with conventional structure are undertaken using Matlab and Ansys. Numerical methods used to represent the nonlinear behaviour of damper and related steps are described. Conjecture regarding the accuracy and adequacy of 2D models to effectively analyse façade panel model subjected to dynamic wind loads are analysed. Different mean wind speeds are considered to compare the performance of the system. The behaviour of the smart dampers subjected to yearly winds is investigated. In particular, the response of the structure subjected to the Sydney yearly wind is carried out to monitor the effectiveness of a smart damper façade system in Sydney. 3D modelling has been used to investigate and subsequently model the system more accurately. As 3D models are always time consuming and need more details, therefore; using this kind of model should have justification. Using simplified wind loads as uniformly distributed load for façade is a rough approximation, although, it is enough to confirm the concept of damper façade system. To come up with a reliable system for installation into the buildings, a more realistic scenario is needed. This type of modelling allows us to see the performance of the system during the wind load which is uncorrelated along the width of the building.

Chapter 5 describes the behaviour of nonlinear double skin façade using Matlab software. The MDOF model presented in this thesis is representing a simplified model which is used in order to demonstrate the behaviour of the proposed system. The complex primary structure with an outer skin facade could be modelled as a two degree of freedom system, where primary mass represents the structure (including the inner skin mass) and the secondary mass represents the outer skin. Usually this kind of Modelling is used to present a tuned mass damper (TMD) system, although there is a different mechanism to apply the load in these cases.

Chapter 6 cover a thorough sensitivity analysis regarding the behaviour of smart damper with different characteristics of wind. Sensitivity analyses are carried regarding variations in stiffness ratio. Four different ratios are chosen to cover most probable and achievable stiffness ratios. Multi linear behaviour damper are used to control the large displacement of the panels and also achieve a similar reduction in the response of the main structure. Low stiffness part in the façade damper behaviour system is unavoidable and leads the system to behave like a multi tuned mass damper system. Previous researchers show that the frequency of the movable facade should be around half of the main structure, frequency which needs a relatively low stiffness connections between façade skin and the main structure. Therefore, the sensitivity analysis is presented to show the optimum secondary stiffness needed for the system to remain effective.

Chapter 7 covers the financial aspects of the study. This financial study is not describing all concepts that have been developed, simulated and analysed during the proof of concept phase, but emphasizes uniquely on demonstrating the feasibility and potentiality for business applications. The numerical simulation results are presented and compared with the building with fixed facade, to highlight the benefits and improvements of the building having flexible/energy absorbing façade.

Here, it should be clarified that the proof of the financial benefits is done using simplified numerical models: the presented results accuracy should, therefore, be taken with caution - they represent the magnitude of the relative costs and they are useful to verify the concept's feasibility and to quantitatively compare different concepts.

Finally, significant conclusions from the research are presented in chapter 8 together with recommendations for future work.

Chapter 2

2 Literature Review on Facade Systems

2.1 Introduction

Race towards building skyscrapers has not been without its challenges. Unfortunately, the emergence of super tall buildings are often accompanied by increased structural flexibility and lack of adequate inherent damping in building structures. In addition, it leads to increase in structure's susceptibility to the actions of winds and other dynamic loads. For taller buildings, impacts of lateral forces become much greater. Fundamental vibration period of structures has a direct correlation with height and taller buildings usually experience longer periods of vibration which is beneficial against earthquake loads but not wind forces. Typical wind force frequencies are between 0.1 to 1 Hz, therefore, they affect a wide range of structures from mid-rise to high-rise structures. Thus, in the case of mid-rise to high-rise structures, resonance conditions can happen during strong storm activities.

Various state-of-the-art technologies are now available and can be used in the design and construction of new buildings in order to improve their dynamic behaviours. Acceptance of innovative systems in building structures depends on a combination of performance enhancement versus construction costs and long-term effects. New innovative devices need to be integrated into these structures, with realistic evaluation of their performance and impact on the structural system as well as verification of their ability for long-term operations.

One of these approaches includes adding energy absorbers to a structure. The aim of involving energy absorbers in a structure is to add damping and reduce wind-induced response of the structure. Correctly implemented, a perfect damper should be able to simultaneously decrease both stress and deflection in the structure. So, increasing the damping ratio by any devices like damper systems is a desirable solution in comparison with costly stiffening systems such as belt truss and out riggers which increase not only stiffness but also mass of the structure to a large extent.

It should be noted that environmental loads are likely to have huge bearing on the vulnerability of exterior face of façade systems and load transmission to the primary structure. Moreover, due to the significant usage of glass in buildings in recent decades, an increasing emphasis has been placed on protecting them during typhoon events. This emphasis and focus on safety of façade systems are due to their high cost of installation, repair and reduction of potential for posing severe safety risks to people during typhoon activities (Connor 2003). It should also be noted that loss of valuable and prime space coupled with initial cost of installing large sized damper systems such as tuned mass dampers, has been accepted unwillingly by building owners and any viable alternative system, without the loss of lettable spaces, to dissipate wind forces will be welcomed by them. The primary focus of this research is, therefore, on evaluating the effectiveness of a smart multi-skin façade system in reducing wind-induced excitation.

2.2 Façade Systems

A 'façade' generally envelops the exterior sides of a building. The word has its roots in the French language, literally meaning "frontage" or "face". The aluminium frame which consists of mullions and transoms is normally in-filled with glass which provides an architecturally pleasing skin as well as advantages such as natural day lighting. From the architectural viewpoint, façade of a building is very important from the design standpoint as it sets the character of the rest of the building. Façade also provides shielding against environmental factors like wind or rain and provides light and ventilation to the structure. Fast developments during the 19^{th} century, in the middle of industrial revolution, led to major advances in construction technologies. In the field of façade systems, these advances resulted in usage of industrialised components in their installation. Also, size of façade components and their strength and durability have been improved. After significant changes in the field of structural design, role of façade systems have become more noteworthy nowadays. Two famous materials which have been used widely since the 1930's are precast concrete and aluminium. After World War II, when usage of façade in buildings had come to a temporary halt, rapid developments in building materials opened up a new view of the façade. Construction of façade increased significantly and reached an incredible boom during mid and late sixties (Streicher et al. 2007). Façade panel is an expensive part of a building construction which amounts to about 20% of the total building cost. Special attention should be paid to its protection from damage or collapse.

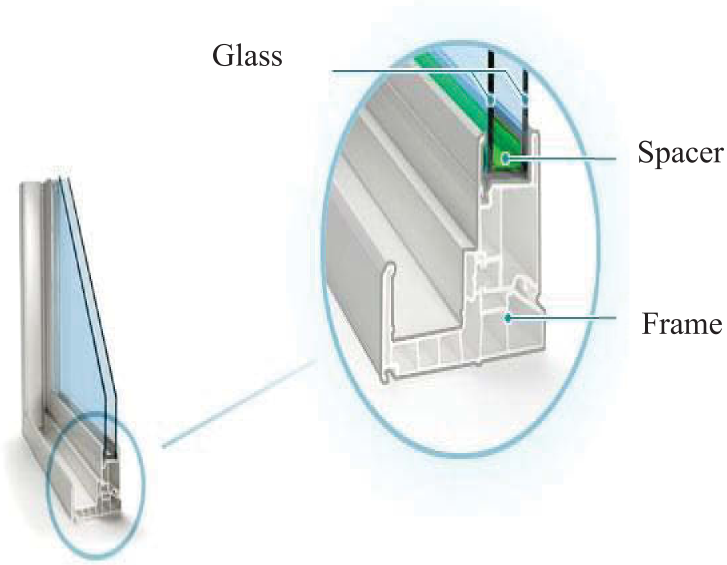


Figure 2.1 Typical components of a façade panel (Milgard)

In modern skyscrapers, exterior walls are often suspended from the concrete floor slabs. Examples include curtain walls and precast concrete walls. In general, the façade systems that are suspended or attached to the precast concrete slabs are made of aluminium (powder coated or anodized) or stainless steel. Typical glazing panels consist of these elements as shown in detail in Figure 2.1:

- Frame: Aluminium frame typically consists of horizontal components, which are called transom, and vertical components which are called mullions.
- Glass: Air or gas fills between the two panes of glass space. Special Low-E coating on the glass blocks infrared light to keep heat inside in the winter and outside in the summer. It also filters damaging ultraviolet light (UV) to help protect interior furnishings from fading.
- Spacer: A spacer keeps a window's dual glass panel in the correct distance apart for optimal airflow between panes. Too much or too little airflow can affect the insulation efficiency of the glass. The design and material of the spacer can also make a big difference in the ability to handle expansion and contraction and thus reduce condensation. Insulating spacers between the panes of glass reduce heat transfer and condensation.

2.2.1 Curtain Walls

Curtain wall is a kind of barrier which separates the exterior of a building from its interior. They hang of load bearing elements such as slabs and are not designed to carry no portion of the gravity loads associated with the main structure. It plays a vital role related to the aesthetic appeal of the primary building and has the following crucial rules:

- Wind/rain/water protection

- Insulation against hot and cold climates
- Protection from noise and pollution

Curtain walls are designed to span multiple floors and take into consideration design requirements such as thermal expansion and contraction, building sway and movement, water diversion and thermal efficiency for cost-effective heating, cooling and lighting in the building. Framed glazed curtain walls are typically designed with extruded aluminium members, although early curtain walls were made of steel. Curtain walls can be divided to the following groups:

2.2.1.1 Stick System

Stick curtains are very common and versatile and can be used for any kind of building from glass high-rise to single storey shop fronts. Because of the number of joints in stick curtain walling, they are generally very good at accommodating variability and movement in the building frame. They are also suitable for irregularly shaped buildings. Assembly is slow compared with pre-assembled systems (Permasteelisa 2009). Figures 2.2 illustrates the stick system façade during the construction, and Figure 2.3 shows a typical assembly of stick system.

Figure 2.2 Stick system façade (Permasteelisa 2009)

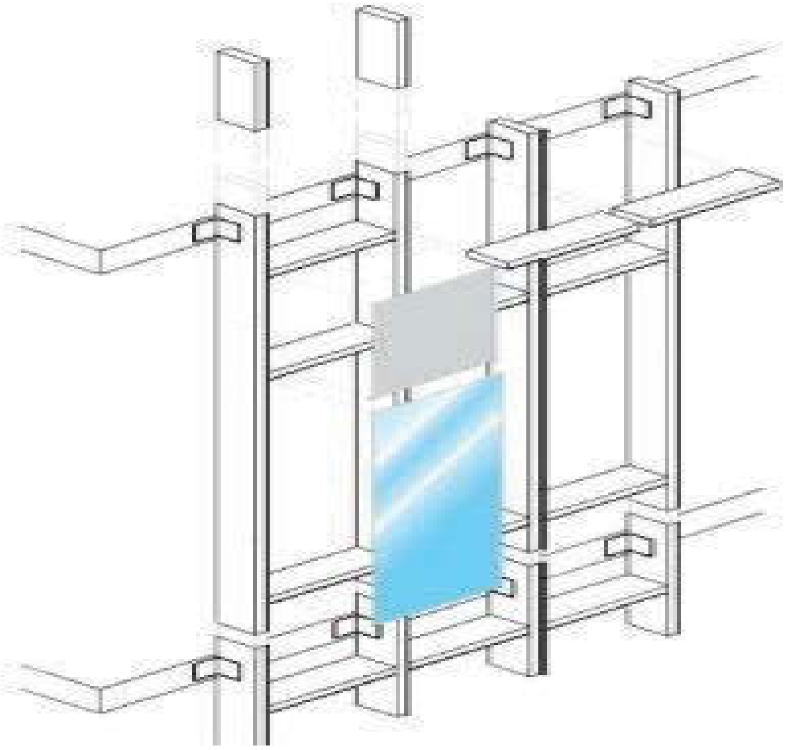


Figure 2.3 Typical assembly of stick system façade (Permasteelisa 2009)

A stick system consists of a framework of horizontal and vertical framing members. Into the framework, infill units are fitted and may comprise a mixture of fixed and opening glazing and insulated panels. The elements are prepared at the plant and afterwards assembled on site as a kit of parts. The mullions are typically spaced between 1.0 and 1.8m. The glazed or opaque panel is retained with a pressure plate or clamping element and screw-fixed every 150 to 300 mm. Sometimes, hammer-in structural gaskets are used instead. The pressure plate is mostly covered with a snap-on decorative element.

2.2.1.2 Unitized Curtain Wall

Unitized curtain walls are pre-fabricated; so mechanical handling is required to position, align and fix the units onto pre-positioned brackets attached to the concrete floor slab or structural frame. They are spanned from floor to floor and are anchored to the building's load-bearing structure. As Figure 2.4 shows pre-fabrication of this type of façade allows for elevated quality controls, makes installation very quick, does not require the use of scaffolding and minimizes work at the worksite with lower installation costs.

Figure 2.4 Unitized curtain wall (Permasteelisa 2009)

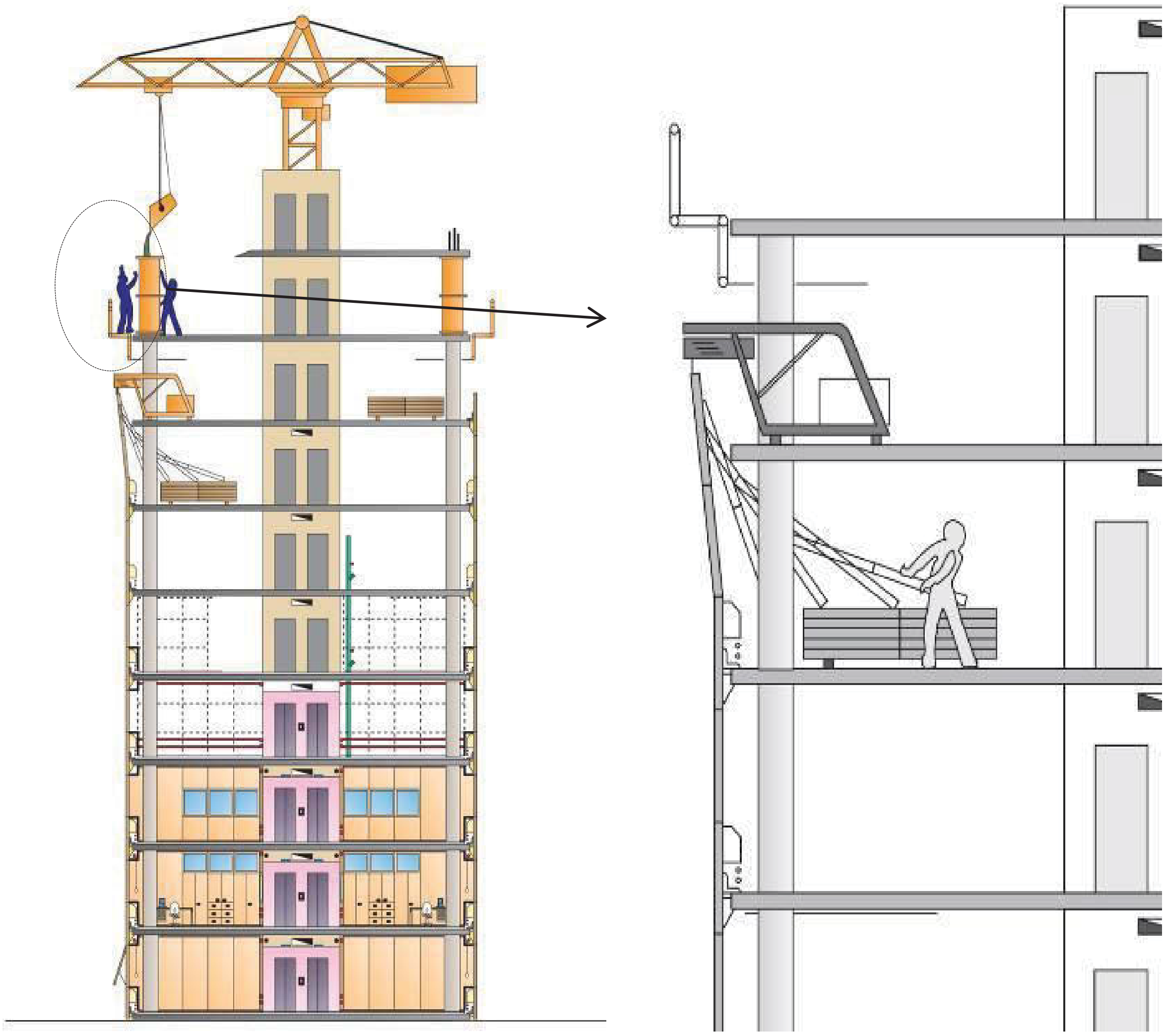


Figure 2.5 Installation of curtain wall (Permasteelisa 2009)

The system is more complex in terms of frame design in comparison to stick system and possibility of creating complex and/or irregular surfaces is limited as shown in Figure 2.5. They have higher direct costs and are less common than stick systems. Fewer site staff is needed in comparison with stick systems and can make the system more cost effective .

2.2.1.3 Spandrel Panel Ribbon Glazing

Spandrel panel ribbon glazing is a long or continuous run of vision units fixed between spandrel panels supported by vertical columns or the floor slabs**.** They can be used in conjunction with spandrel panels, that is, horizontally spanning prefabricated or precast concrete units. It may also be used with spandrels comprised of stand walls faced with rain screen panels. An example of spandrel panel during construction is shown in Figure 2.6.

Figure 2.6 Example of spandrel panel ribbon glazing (Permasteelisa 2009)

2.2.1.4 Panelized Curtain Wall

Panellised curtain walling comprises large prefabricated panels of bay width and storey height, which are connected back to the primary structural columns or to the floor slabs close to the primary structure. Fixing the panels close to the columns reduces problems due to deflection of the slab at mid span which affects stick and unitised systems. Panels may be of precast concrete or comprise a structural steel framework which can be used to support most cladding materials (e.g. Stone, metal and masonry). Figure 2.7 shows the installation procedure of penalizing curtain wall.

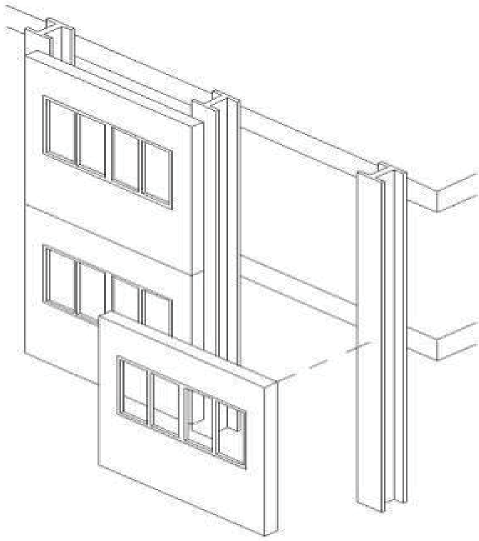


Figure 2.7 Penalized curtain wall (Permasteelisa 2009)

2.2.1.5 Bolted Glass Façade

This type of façade was created to fulfil an architectural and functional requirement for maximum transparency. It eliminates all opaque supporting elements and does not employ sticks. Glazed panels are suspended instead using the lightest possible systems available. There are two different types of glass façade: the independent assembly and the suspended assembly, which are shown in Figures 2.8 and 2.9.

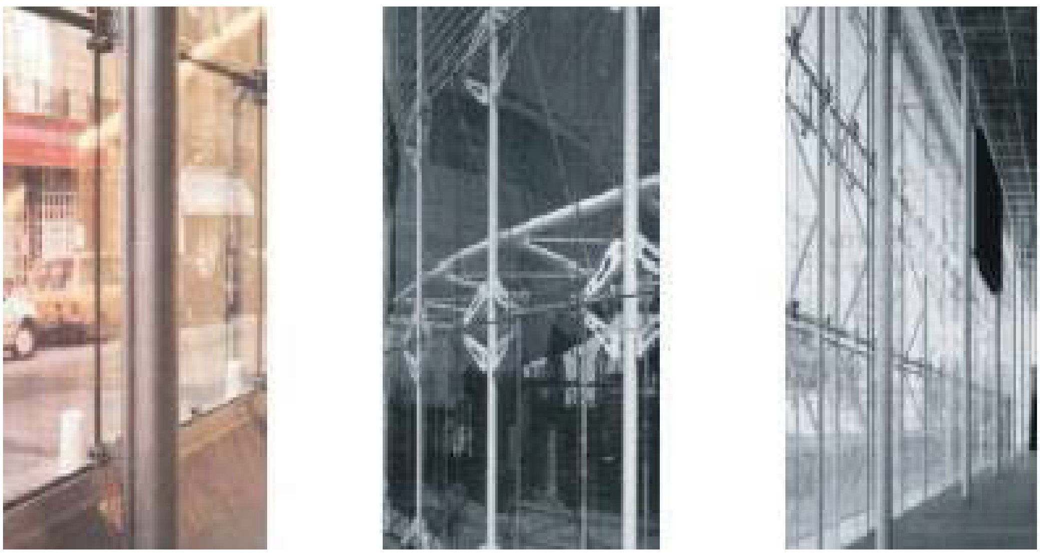


Figure 2.8 Independent assembly (Permasteelisa 2009)

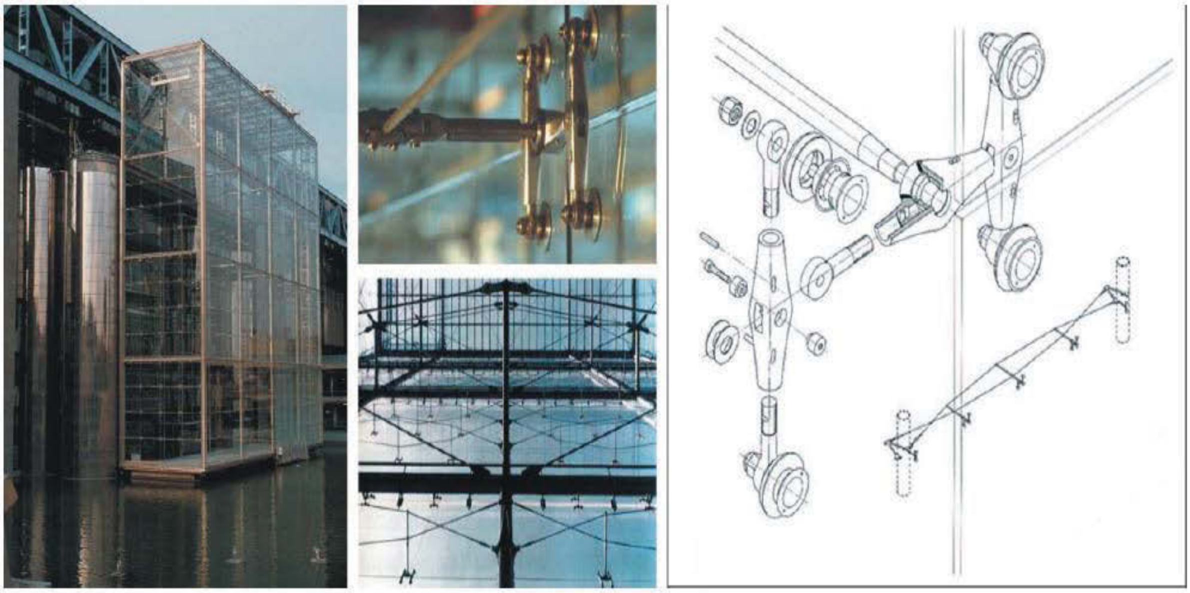


Figure 2.9 Suspended assembly (Permasteelisa 2009)

2.2.1.6 Double Skin Façade (DSF)

2.2.1.6.1 Definition

Developments of a new kind of façade system have been boosted because of energy performance concerns of previous façade technologies. The attention of government authorities and building owners to environmentally friendly structures has encouraged these state-of-the-art concepts. Transparency and visual attraction are other crucial factors which have led to tall glass skyscrapers. Based on those essential elements, double skin or multi skin façade (also known as active envelope) systems are recently presented as a viable solution for achieving the aforementioned goals in modern architectures. They consist of two or more panes which are separated by a cavity through which air can circulate naturally or mechanically.

In most cases, a shading device is provided in the cavity (Hensen, Bartak & Drkal 2002). High costs associated with complexity of design and installation can be justified by their increasing demands because of wide transparent surfaces and high thermal performance. The double-skin façade systems are an architectural concept driven by desire for aesthetics leading to mostly all-glass high rise buildings. For evaluation of installation of double skin façade as a glazing envelope for buildings, the following factors should be considered: climate, orientation, detailing, and construction cost as well as the energy price. They should be evaluated precisely (Poirazis 2004a). Double Skin Façades can be described as a traditional single façade doubled inside or outside by a second and essentially glazed façade. Apart from the type of ventilation inside the cavity, the origin and destination of the air can differ depending mostly on climatic conditions, usage, location, occupational hours of the building and the heating, ventilation and air conditioning (HVAC) strategy. Each of these two façades is commonly called a skin.

These skins are placed in such a way that air flows in the intermediate cavity (Poirazis 2004b). Through the cavity, for example, hot air can be effectively removed in summer time and also natural ventilation can be introduced even at higher levels of tall buildings because there are additional exterior skins which act as wind buffers. These functions of the DSF system reduce energy usage during building operation, potentially resulting in economic benefits in the long run, even though their initial construction cost is higher than that of conventional single skin façades. The glass skins can be single or double glazing units with a distance of 20 cm to 2 m between them. often, for protection and heat extraction reasons during the cooling periods, solar shading devices are positioned inside the cavity (Saelens, Roels & Hens 2003).

DSFs have the potential to reduce building heating and cooling energy consumption in several ways. However, not all DSFs built in recent years perform well. Furthermore, in most cases, large air-conditioning systems have to make up for summer overheating problems and energy consumption often exceeds the intended heating energy savings. Other concerns about DSF performance include fire safety (fires spreading between floors via the cavity) and their costly maintenance (Streicher et al. 2007).

2.2.1.6.2 History of Façade Systems

Jean-Baptiste Jobard, director of the Industrial Museum in Brussels, described an early version of a mechanically ventilated multiple skin façade in 1849 (Poirazis 2004b). He mentioned how in winter, hot air should be circulated between two glazing while, in summer, it should be cold air (Saelens, Roels & Hens 2004). The first double skin curtain wall appeared in 1903 in the Steiff factory in Giengen/ Brenz. Priorities were to maximize day lighting while taking into account the cold weather and strong winds of the region (Saelens et al. 2005). The solution was a three-storey structure with a ground floor for storage space and two upper floors were used for work areas.

The structure of the building proved to be successful and two additions were built in 1904 and 1908 with the same double skin system but using timber instead of steel in the structure for budgetary reasons (Streicher et al. 2007). In Russia, Moisei Ginzburg made an experiment with double skin strips in the communal housing blocks of his Narkomfin building (1928) and Le Corbusier designed the Centrosoyus in Moscow (Poirazis 2004b). A year later, Le Corbusier started the design for the Cite de Refuge (1929) and the Immeuble Clarte (1930) in Paris and postulated two new features. Little or no progress was made in double skin glass construction until the late 1970s and early 1980s. During the 1980s, this type of façade started gaining momentum.

Most were designed while considering environmental concerns, like the offices of Leslie and Godwin. In other cases, the aesthetic effect of multiple layers of glass was the principal concern. In the 1990s, two factors strongly influenced the proliferation of DSFs. Environmental concerns started influencing architectural design, both from a technical standpoint and as a political influence that made 'green buildings' a good image for corporate architecture (Braham 2005).

2.2.1.6.3 Examples

Examples of notable buildings which utilise a skin facade are the 30 St Mary Axe (also known as The Gherkin) and 1 Angel Square. Both of these buildings achieved great environmental credentials for their size, with the benefits of a double skin being a key to this (Figure 2.10 and 2.11). The Gherkin features triangular windows on the outer skin which skelter up the skyscraper (Poirazis 2004b). These windows open according to weather and building data, allowing more or less air to cross flow through the building for ventilation (Roth, Lawrence & Brodrick 2007).

Figure 2.10 Suspended assembly (Permasteelisa 2009)

Figure 2.11 Suspended assembly (Permasteelisa 2009)

Chapter 3

3 Characteristics of wind loads and methods of mitigating wind effects

3.1 Introduction

The wind loads are relevant for both structural frames and components/cladding. The former are for the design of structural frames, such as columns and beams. The latter is for the design of finishings and bedding members of components/cladding and their joints. Wind loads on structural frames and on components/cladding are different, because there are large differences in their sizes, dynamic characteristics and dominant phenomena and behaviours. Wind loads on structural frames are calculated on the basis of the elastic response of the whole building against fluctuating wind forces. Wind loads on components/cladding are calculated on the basis of fluctuating wind forces acting on a small part. Wind resistant design for components/cladding has been inadequate until now. They play an important role in protecting the interior space from destruction by strong wind. Therefore, wind resistant design for components/cladding is as important as for structural frames.

3.2 Classification of Wind Load

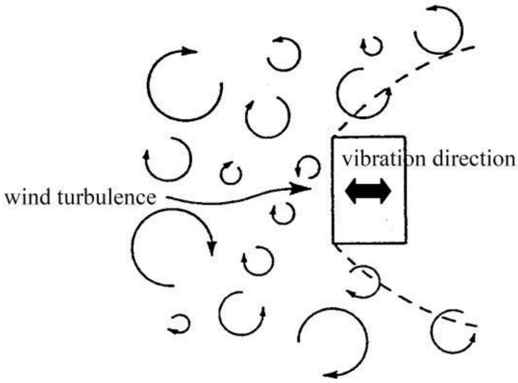
A mean wind force acts on a building. This mean wind force is derived from the mean wind speed and the fluctuating wind force produced by the fluctuating flow field. The effect of the fluctuating wind force on the building or parts thereof depends not only on the characteristics of the fluctuating wind force but also on the size and vibration characteristics of the building or part thereof. Therefore, in order to estimate the design wind load, it is necessary to evaluate the characteristics of fluctuating wind forces and the dynamic characteristics of the building (See Figure 3.1).

The following factors are generally considered in determining the fluctuating wind force.

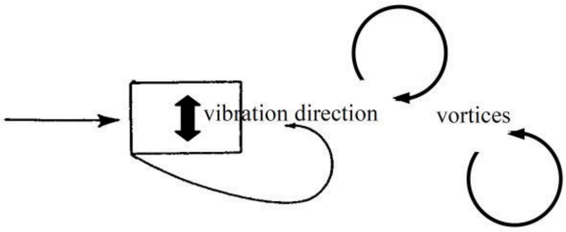
1) Wind turbulence (temporal and spatial fluctuation of wind)

2) Vortex generation in wake of building

3) Interaction between building vibration and surrounding air flow



a) Fluctuating wind forces caused by wind turbulence



b) Fluctuating wind forces caused by vortex generation in the wake of building

Figure 3.1 Fluctuating wind forces based on wind turbulence and vortex generation in the wake of building

Fluctuating wind pressures act on building surfaces due to the above factors. Fluctuating wind pressures change temporally, and their dynamic characteristics are not uniform at all positions on the building surface. Therefore, it is better to evaluate wind load on the structural frames based on overall building behaviour and that on components/cladding based on the behaviour of individual building parts. For most buildings, the effect of fluctuating wind force generated by wind turbulence is predominant. In this case, horizontal wind load on the structural frames in the along-wind direction is important. However, for relatively flexible buildings with a large aspect ratio, horizontal wind loads on the structural frames in the crosswind and torsional directions should not be ignored.

For roof loads, the fluctuating wind force caused by separation flow from the leading edge of the roof often predominates. Therefore, the wind load on structural frames is divided into two parts: horizontal wind load on structural frames and roof wind load on structural frames.

3.3 Combination of Wind Loads

Wind pressure distributions on the surface of a building with a rectangular section are asymmetric even when the wind blows normal to the building surface. Therefore, wind forces of the crosswind and torsional effects are not zero when the wind force in the along-wind direction is a maximum. The combination of wind loads in the along-wind, across-wind and torsional directions have not been fully taken into consideration so far, because the design wind speed has been used without considering the effect of wind direction. However, with the introduction of wind direction, the combination of wind loads in the along-wind, across-wind and torsional directions becomes necessary. Hence, it has been decided to adopt explicitly a mechanism for the combination of wind loads in along-wind, across-wind and torsional directions.

3.4 Wind Directionality Factor

Occurrence and intensity of wind speed at a construction site vary for each wind direction with geographic location and large-scale topographic effects. Furthermore, the characteristics of wind forces acting on a building vary for each wind direction. Therefore, rational wind resistant design can be applied by investigating the characteristics of the wind speed at a construction site and wind forces acting on the building for each wind direction. These recommendations introduce the wind directionality factor in calculating the design wind speed for each wind direction individually. In evaluating the wind directionality factor, the influence of typhoons, which is the main cause of strong winds in countries like Japan, should be taken into account. However, it was difficult to quantify the probability distribution of wind speed due to a typhoon from meteorological observation records over only 70 years, because the occurrence of typhoons hitting a particular point is not necessarily high. In these recommendations, the wind directionality factor was determined by conducting Monte Carlo simulation of typhoons, and analysis of observation data provided by the Metrological Agency.

3.5 Reference Height and Velocity Pressure

The reference height is generally the mean roof height of the building, as shown in Figure 3.2. The wind loads are calculated from the wind pressure at this reference height. The increasing vertical distribution of wind load is reflected in the wind force coefficients and wind pressure coefficients. The wind load for a lattice type structure, for example, shall be calculated from the wind pressure at different heights, as shown in Figure 3.2.

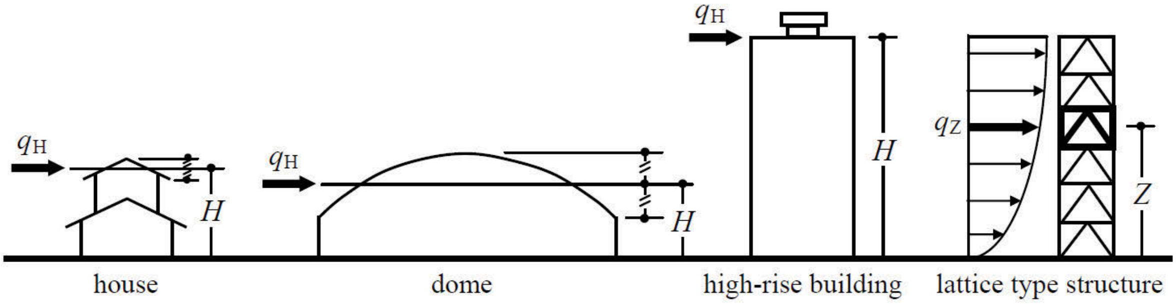


Figure 3.2 Definition of reference height and velocity pressure

3.6 Wind Load on Structural Frames

The maximum loading effect on each part of the building can be estimated by the dynamic response analysis considering the characteristics of temporal and spatial fluctuating wind pressure and the dynamic characteristics of the building. The equivalent static wind loads producing the maximum loading effect is considered as the design wind load. For the response of the building against strong wind, the first mode is predominant and higher frequency modes are not dominant for most buildings. The horizontal wind load (along-wind load) distribution for structural frames is assumed to be equal to the mean wind load distribution, because the first mode shape resembles the mean building displacement. Specifically, the equivalent wind load is obtained by multiplying the gust factor, which is defined as the ratio of the instantaneous value to the mean value of the building response, to the mean wind load. The characteristics of the wind force acting on the roof are influenced by the features of the fluctuating wind force caused by separation flow from the leading edge of the roof and the inner pressure, which depends on the degree of sealing of the building. For example a short afterbody will lead to full suction acting on the roof while a long afterbody will mpst probably lead to suction at the leading edge and positive pressure near the leeward edge due to reattachment of wind. Therefore, the characteristics of roof wind load on structural frames are different from those of the along-wind load on structural frames. Thus, the roof wind load on the structural frames cannot be evaluated by the same procedure as for the along-wind load on structural frames. Here, the gust factor is given when the first mode is predominant and assuming elastic dynamic behaviour of the roof structure under wind load.

3.7 Wind Load on Components/Cladding

In the calculation of wind loads on components/cladding, the peak exterior wind pressure coefficient and the coefficient of interior wind pressure are considered, and the peak wind force coefficient is calculated as their difference. Only the size effect is considered. The resonance effect is ignored, because the natural frequency of components/cladding is generally higher than the frequency of fluctuating winds. The wind load on components/cladding is prescribed as the maximum of positive pressure and negative pressure for each part of the components/cladding for wind from every direction, while the wind load on structural frames is prescribed for the wind direction normal to the building face. Therefore, for the wind load on components/cladding, the peak wind force coefficient or the peak exterior wind pressure coefficient must be obtained from wind tunnel tests or other verification methods.

3.8 Wind Loads in a Crosswind and Torsional Directions

It is difficult to predict responses in the crosswind and torsional directions theoretically like along-wind responses. However, an empirically based prediction formula is given in current recommendations based on the fluctuating overturning moment in the crosswind direction and the fluctuating torsional moment for the first vibration mode in each direction.

3.9 Vortex Induced Vibration and Aeroelastic Instability

Vortex-induced vibration and aeroelastic instability can occur with flexible buildings or structural members with very large aspect ratios. Criteria for a crosswind and torsional vibrations are provided for buildings with rectangular sections. Criteria for vortex-induced vibrations are provided for buildings and structural members with circular sections. If these criteria indicate that vortex-induced vibration or aeroelastic instability might occur, structural safety should be confirmed by wind tunnel tests and so on. A formula for wind load caused by vortex-induced vibrations is also provided for buildings or structural members with circular sections.

3.10 Small-scale Buildings

For small buildings with large stiffness, the size effect is small and the dynamic effect can be neglected. Thus, a simplified procedure is employed.

3.11 Effect on Neighbouring Buildings

When groups of two or more taller buildings are constructed in proximity to each other, the wind flow through the group may be significantly altered and cause a much more complex effect than is usually acknowledged, resulting in higher dynamic pressures and motions, especially in neighbouring downstream buildings.

3.12 Assessment of Building Habitability

Building habitability against wind-induced vibration is usually evaluated on the basis of the maximum response acceleration for usually 1-year-recurrence wind speed. Hence, these recommendations show a map of 1-year-recurrence wind speed based on the daily maximum wind speed observed at meteorological stations and a calculation method for response acceleration.

3.13 Shielding Effect by Surrounding Topography or Buildings

When there are topographical features and buildings around the construction site, wind loads or wind-induced vibrations are sometimes decreased by their shielding effect. Rational wind resistant design that considers this shielding effect can be performed. However, changes to these features during the building's service life need to be confirmed. Furthermore, the shielding effect should be investigated by careful wind tunnel study or other suitable verification methods, because it is generally complicated and cannot be easily analysed.

3.14 Wind Characteristics

An important characteristic of wind is the variation of speed with height. The local mean velocity, which is referred to simply as "wind speed", is zero at the surface, and it increases with height above ground in a layer, within approximately 1 kilometre from the ground, called the "atmospheric boundary layer". Above this layer, the wind speed is essentially constant, as shown in Figure 3.1, up to a height where the ceiling boundary layer is encountered.

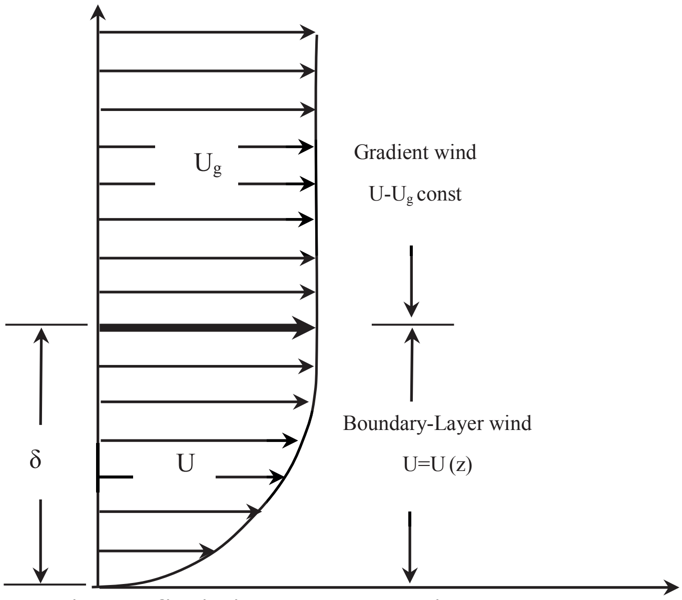


Figure 3.3Wind velocity profile in ideal atmospheric boundary layer

Wind is an atmospheric turbulent flow, characterized by the random fluctuations of velocity and pressure. If the instantaneous velocity of wind at a given point is recorded as a function of time on a chart, the result will look like that in Figure 3.4. The wind near ground level is highly turbulent. Its velocity vector V at any time can be decomposed into three components, namely, longitudinal, lateral, and vertical. Each component can further be decomposed into a mean and a fluctuating part as follows:

$$Longtudinal\ component = U + u$$

$$Lateral\ component = V + v$$

$$Vertical\ component = W + w$$

Where the temporal averages are denoted by *U*, *V,* and *W* and the lowercase letters denote time dependent fluctuating components. Representative traces of the *longitudinal component* = *U* + *u* is shown in Figure 3.4.

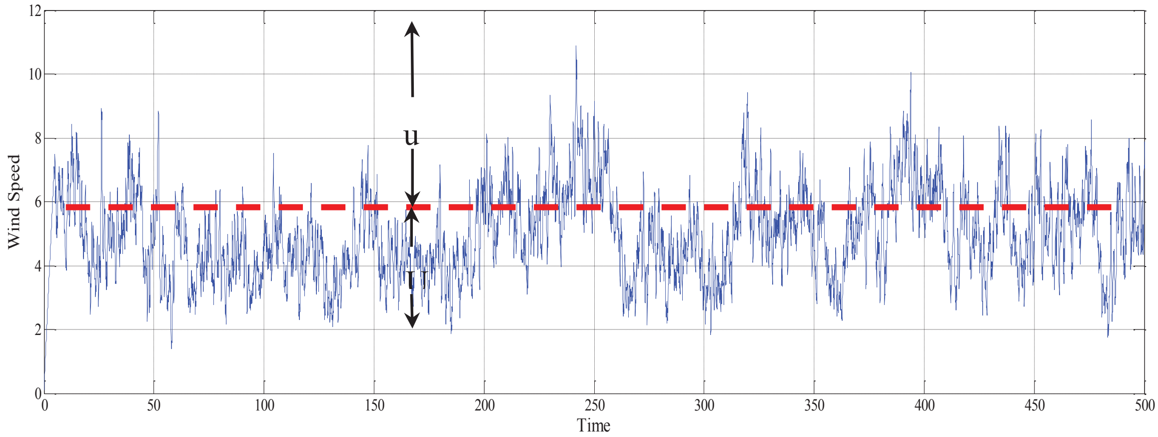


Figure 3.4Typical trace of longitudinal wind speed

Atmospheric turbulence is always three-dimensional even if the mean velocity of flow is one- or two-dimensional. For instance, although the wind over a large flat area is essentially horizontal (V = O and W = 0), all three components of turbulence u, v, and w exist. Information on the features of atmospheric turbulence is useful in structural engineering applications for three main reasons.

First, rigid structures and members are subjected to time-dependent loads with fluctuations due in part to atmospheric turbulence.

Second, flexible structures may exhibit resonant amplification effects induced by velocity fluctuations.

Third, the aerodynamic behaviour of structures—and, correspondingly, the results of tests conducted in the laboratory may depend strongly upon the turbulence in the air flow.

The following features of the atmospheric turbulence are of interest in structural applications: the turbulence intensity; the integral scales of turbulence; the spectra of turbulent velocity fluctuations; and the cross-spectra of turbulent velocity fluctuations. Also of interest to structural designer is the dependence of the largest wind speeds in a record upon averaging time (Simiu & Scanlan 1986a).

3.14.1 Wind-Excited Motion of Tall Buildings

It has been substantiated by (Davenport 1967), (Vickery & Basu 1983), (Van Koten 1967), and others that the wind action and the resultant building motion are statistical and dynamic phenomena. The motion of tall buildings has been observed to occur primarily in three modes of action: along-wind, cross-wind, and torsional modes. As an example, for a rectangular building with one face nearly perpendicular to the mean flow, the motion has been measured in the along-wind and cross-wind directions, as well as in the torsional mode. Each mode of vibration will be briefly reviewed below.

3.14.2 Along-Wind Motion

The along-wind response of buildings due to buffeting by atmospheric boundary-layer turbulence has been studied by many investigators (Davenport, Hogan & Vickery 1970); and (Solari 1993). Based on these studies, (Solari 1983) developed a set of closed-form solutions to estimate the first mode of vibration of both rectangular structures (buildings) and small elevated blocks (point-like structures).

Building motion in the direction of the mean wind can be considered to consist of a mean component corresponding to the mean wind, and a fluctuating component corresponding to the "gust" of the turbulent wind ((Davenport, Hogan & Vickery 1970); (Van Koten 1967)).

Davenport has emphasized that the fluctuating component of the building motion can then be conveniently divided into that part responding to wind frequency components significantly lower than the building natural frequency; and to that part exhibiting a resonance response. The ratio of this "background" response to resonant response depends on the relation between the geometric and dynamic properties of the building to those of the turbulent natural wind. So in different situations either of these dynamic phenomena may dominate.

Motion and loading in the along-wind direction has been shown to be satisfactorily treated by the "Gust Factor Approach". The Gust Factor procedure takes into consideration the following:

1. The exposure of the building to the local wind environment;

2. The dynamic and geometric properties of the building to determine "response factor"; and,

3. Coefficients from model measurements in wind tunnel boundary layers which simulate the turbulent characteristics of strong winds.

Relative to the traditional quasi-static approach, this method is a substantial improvement in method and as such has given a substantial improvement in the accuracy of along-wind prediction because of its more fundamental approach.

Currently, to achieve any improvement in the Gust Factor approach, the use of the wind tunnel is required to model the building and its environment. This is often complemented by a special meteorological study. Using the wind tunnel, such factors as the response of the building to wind direction, topographic effects, the probability of extreme winds and deflections, the stress history, and the detailed dynamics of the building can be studied in considerable detail.

3.14.3 Cross-Wind Motion

Some structures are very sensitive to motion transverse to the mean wind direction if they are uniformly prismatic in cross-section. (This structural response is normally described as "cross -wind" motion, but defined in the principal body axis at small angles of attack, rather than being restricted to motion perpendicular to the wind.) The sensitivity of the cross-wind motion may be particularly apparent as the wind speed increases. It has also been shown in tall buildings such as the John Hancock building in Chicago, that the motion transverse to the mean wind dominates over the along-wind motion at small angles of attack (Davenport, Hogan & Vickery 1970).

Cross-wind motion of buildings is caused by the combined forces from three sources:

1) Buffeting by turbulence in the cross-wind direction,

2) Vortex shedding, and

3) Aero-elastic phenomena such as lock-in or galloping. Due to the complex interaction among these forces, current knowledge in this field is incomplete. All available methods for predicting cross-wind motion rely heavily on wind-tunnel data.

In the Hancock building, when the mean wind speed at the top of the building was estimated at 46 km/hr, the standard deviation of the cross-wind building motion was about three times the along-wind value. For less than a 40% increase in mean wind velocity to 63 km/hr, the standard deviation had risen until the cross-wind motion was nine times the along-wind motion.

In the cross-wind direction, the mean response is near zero but the fluctuating response is magnified greatly from the fluctuating applied load, and in fact the peak response may be greater in the cross-wind direction than in the along-wind direction. This phenomenon is well known to aerodynamics engineers. It has been widely documented (Melbourne 1975), (Saunders & Melbourne 1975); (Reinhold & Sparks 1979); (Kareem 1992); (Kwok 1982). And (Dalgliesh, Cooper & Templin 1983), although many practising structural engineers acknowledge this as a surprise.

The occupants of a tall building can be disturbed by excessive levels of the fluctuating component, i. e., the acceleration and effects of the motion on the building (Robertson & Chen 1967); (Reed 1971); (Chen & Robertson 1972); (Hansen, Reed & Vanmarcke 1973)). So in designs of taller buildings, vibration may dominate the design.

A design approach to cross-wind loading with the degree of practicality and reliability of the Gust Factor approach for along-wind loading has not been possible to date. This has been because the cross-wind vibration at the higher wind velocities is often due to the phenomena called “vortex shedding” and “galloping” whose excitation of the building has not yielded to simple models.

Galloping has been extensively investigated at a fundamental level by many researchers (Parkinson & Brooks 1961), (Parkinson & Smith 1964), (Novak 1969) and (Davenport, Hogan & Vickery 1970); and (Parkinson & Wawzonek 1981). Vortex shedding has been investigated widely in the general fluid mechanics field. With regard to studies akin to tall buildings, principally there have been many studies including those by (Scruton 1967) and (Vickery & Basu 1983). The aerodynamic mechanisms of galloping and vortex shedding are not discussed here. For the details readers are directed to the references mentioned above.

Rosati (1971) has considered representative model buildings and their environments, and developed empirical formulae and curves for these modelling situations (Lawson 1971) emphasized that the lack of ability to determine the likely acceleration levels at the early design stages and thereby the required minimum stiffness of the building, is of some concern and inconvenience.

At the preliminary design stage, the building is not normally well defined aeroelastically, but this is the stage when it would be convenient to be able to estimate the acceleration levels. This would then indicate the degree of stiffness and/or damping required for the building.

Currently, building design has to be advanced until it is suitably defined with regard to its geometric and dynamic properties (stiffness, vibration periods, mode shapes and damping) to make wind-tunnel testing efficient. The later in the design stage of a tall building is determined that wind-tunnel tests suggest structural changes, the more expensive the changes become. The estimation of the acceleration levels of most tall buildings in the cross-wind direction can be made using empirical curves, such as the curve proposed by (Vickery 1973); however, the accuracy of these curves has not been verified.

3.14.4 Torsional Motion

Severe distortions due to the combined effects of cross-wind loads and torsional moments occurred during the 1926 Florida hurricane in two Miami high-rise structures, the 15-storey Realty Building, and the 17-storey Meyer-Kiser Building (Schmitt 1926). Both buildings had unusually narrow shapes in plan (the dimensions in plan of the Meyer-Kiser Building were about 14 m * 42 m). Their structural systems consisted of steel frames. The two transverse end frames of the Meyer-Kiser Building experienced horizontal deflections of about 0.60 m and 0.20 m, respectively (Simiu & Scanlan 1986b).

Torsional effects are due to the fact that in any individual building the centre of mass and the elastic centre do not coincide with the instantaneous point of application of the resultant aerodynamic loads. Until recently, relatively little work has been performed on the development of design guidelines and analytical procedures for use by structural designers.

The first attempt at studying analytically, torsion induced on buildings by fluctuating wind loads, was reported by (Parkinson & Wawzonek 1981). More recently, (Foutch & Safak 1981) have presented potentially useful methods for estimating the along-wind, cross-wind, and torsional response of rectangular buildings. However, owing to the absence of sufficient information on aerodynamic loads, the methods are not presently usable for design purposes.

Wind-tunnel and full-scale research studies of torsional response were first reported by (Reinhold & Sparks 1979) and (Hart, Lew & DiJulio 1975). (Reinhold & Sparks 1979) discussed information on wind-induced torsional moments in an isolated square building model having a height to width ratio of h/b = 8.33 in a flow that simulated urban conditions.

In the study of a 153 m moment-resisting frame, Reed (1971) measured the ratios of the torsional, transverse, and along-wind accelerations to be 2.7:1.8:1. This occurred at a wind speed of 72 km/hr at the top of the building and the motion was sufficient to cause significant objection from the occupants. In order to overcome this adverse effect in the design of tall buildings, the use of “tube-in-tube” construction, trussed cantilevering, or more sophisticated means of increasing torsional stiffness, appear to generate a level of torsional rigidity such that the torsional vibration is not normally a problem. However, it should be noted that the amount of full-scale data reporting on the actual motion and modes of vibration of tall buildings is not substantial.

Torsional motion is of special interest to the structural engineer for two reasons in particular. Cladding and their anchors may be damaged by the torsional motion. And also, if a tall building is undergoing torsional motion and occupants are looking out of the building and they line up vertical parts of the windows with distant objects, the apparent motion of the building is amplified (Reed 1971). This visual acuity effect will emphasize the motion of the building over that of rectilinear motion.

Due to the infancy of this field of study, little information for design is available other than data obtained by wind-tunnel tests. The torsional mode of vibration has rarely been modelled in the tunnel by a full aeroelastic model which is necessary for accurate results. Also, the torsional stiffness of a building is not a simple determination. In some buildings, neglecting torsional motion can be a serious oversimplification (Reed 1971).

3.14.5 Wind Records

The von Kármán wind turbulence model, also known as von-Kármán gusts, is a mathematical model of continuous gusts. It matches observed continuous gusts better than the Dryden Wind Turbulence Model and is the preferred model of the United States Department of Defense in most aircraft design and simulation applications. The von-Kármán model treats the linear and angular velocity components of continuous gusts as spatially varying stochastic processes and specifies each component's power spectral density.

The auto spectral density for a longitudinal component of turbulence, according to the von Kármán model, is given as

$$\frac{nS_{uu}(n)}{\sigma_u^2} = \frac{4\tilde{n}_u}{\left(1+70.8\tilde{n}_u^2\right)^{5/5}} \tag{3.1}$$

Where S_{uu} is the auto-spectrum of the wind speed variation, n is the frequency of vibration, $\sigma_u$is the standard deviation of the wind speed variation and $\widetilde{n_u}$ is a non-dimensional frequency parameter given by:

$$\widetilde{n_u} = \frac{n^x L_u}{U} \tag{3.2}$$

Here L_u is the length scale of longitudinal turbulence and U is the mean wind speed. Figure 3.5 shows the wind pressure trend for longitudinal turbulence of 3 and mean wind speed of 22 m/s.

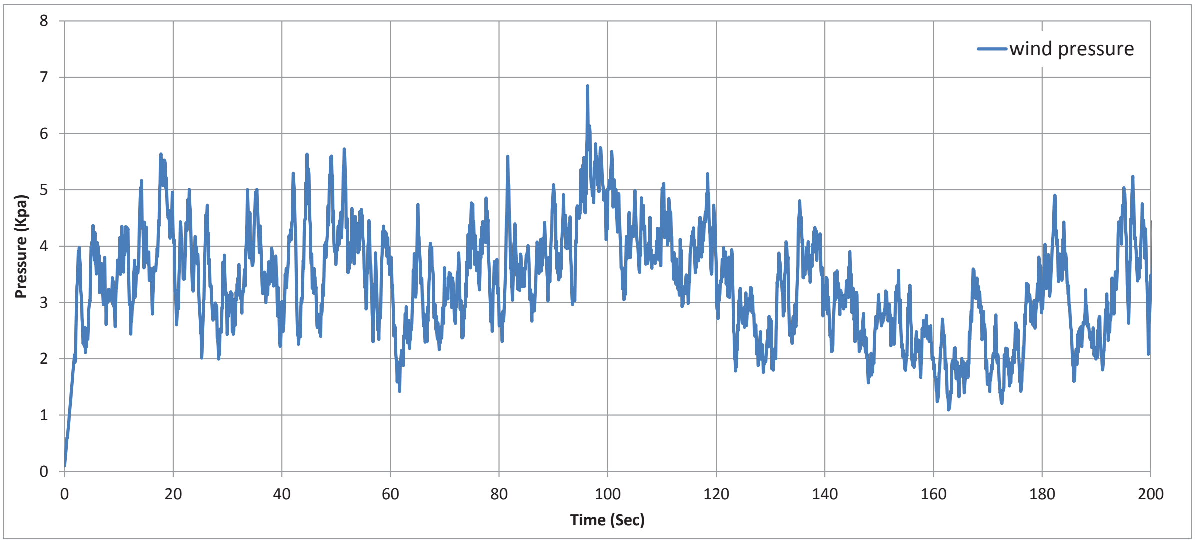


Figure 3.5Wind pressure trend used for analysis

3.15 Means to Reduce Wind-Induced Vibration of Tall Buildings

The wind-induced dynamic response of tall buildings can be controlled by global design modifications that can be placed into four major categories, namely, architectural modifications, modifications in structural systems, cladding isolation, and addition of damping systems. A summary of methods to control wind-induced building motions has been presented by (Kareem, Kijewski & Tamura 1999). The summary was reviewed and is discussed below,

3.15.1 Architectural Modifications

The wind-induced motion of a tall building can be controlled either by reducing the wind loads or by reducing the response. For example, an appropriate choice of building shape and architectural modifications can result in the reduction of motion by altering the flow pattern around a building. Open passages in the building would allow the air to bleed into the wake and separated regions thereby increasing the base pressure and consequently reducing aerodynamic forces.

Nakamura (1975) has reported that there is no definite improvement in the overall response of a two dimensional-prism with holes except for the case when wind approaches at zero angle of attack. Similar results were found by (Kareem 1992) for a building model exposed to a turbulent boundary-layer flow. Furthermore, buildings with tapered and non-uniform cross sections along the height have less potential of creating a coherent wake.

The provision of a series of holes or openings through the upper portion of the building can effectively facilitate disruption of organized vortices which results in reducing wind-induced load effects (Kareem, Kijewski & Tamura 1999).

3.15.2 Modifications in Structural Systems

An efficient structural system can provide the most effective means of controlling structural dynamic response. The use of space frame and mega-frame concepts, outrigger trusses, belt trusses and Band-Aid type stiffening systems can offer additional resistance to wind loads (Kareem 1992). Other alternatives include modification of the structural mode shapes to increase the mass participating in the dynamics of building in the fundamental mode. However, in this situation care must be exercised as the contributing loading in the fundamental loading may also experience an increase. Other options may include shifting the major stiffness axes from the principal geometric axes (Kareem 1992). These solutions are however costly with little possibility of retrofitting a structure through such modifications. Although other conventional technics like mass damper and tunned damper are more likely to be more feasible.

3.15.3 Cladding Isolation

Kareem (1992) proposed the concept of isolation in the mountings of the cladding to the structural system. Buildings are isolated from earthquake excitation by employing isolator bearings between the building and the foundations and a similar concept is proposed for cladding. The integrated effects of the unsteady aerodynamic loads acting on cladding are transferred to the frame which results in building motion.

If the cladding is connected to the frame by an isolation mounting, then the aerodynamic loads transferred to the frame will be reduced and consequently the building motion will be reduced. In order for this mounting to be effective, the ratio of excitation frequency to the natural frequency of the cladding system should be greater than square root of two (Kareem 1992). In this situation the mounting system is more effective without any damping based on principles of vibration isolation.

The proposed system can be materialized by dividing the claddings on the building envelop into several segments. The preliminary calculations of Kareem (1992) suggest that such a mounting system will be quite soft and pneumatic mounts may be an appropriate choice. Such an installation may cause the cost of a cladding system to rise significantly. This can be overcome by using these systems in staggered configurations and the remaining portions of the building envelop may utilize conventional cladding. The staggered arrangement has been proposed to help reduce the correlation of wind-induced pressure which in turn would result in lessening of the integrated loads.

3.15.4 Addition of Damping Systems

Damping is becoming a part of the structural engineers design vocabulary. It is well known that the introduction of damping into structural systems is a very efficient method in reducing the effects of dynamic loads on these systems. Over the past thirty-five years the idea of introducing a separate system to increase the damping in buildings has gained widespread acceptance, McNamara et al. (1997). Wind engineers have introduced damping systems in large scale structures such as the World Trade Centre (Mahmoodi et al. 1987) and Citicorp Center (McNamara & J 1977) in New York city.

In the design of tall buildings, engineers must assume a level of natural damping in the structure in order to assess the building habitability during frequent wind storms. The actual damping in building structures is a difficult quantity to measure prior to building construction and varies with response levels, type of structural systems, cladding system and materials used for construction. Recognizing this uncertainty associated with estimating the natural damping in structural systems, engineers have introduced energy dissipating systems into the design of buildings to augment damping. These systems are designed to provide specific amounts of damping and by controlling the damping provided the uncertainty associated with assuming the damping present is eliminated. Structural designers attempting to solve motion comfort problems in tall buildings have found that direct addition of damping to the structure is the most reliable way of assuring a well-behaved structure in turbulent environments (McNamara et al. 1997).

The use of energy dissipating systems related to wind effects on buildings is focused on the reduction of the acceleration response of the upper floors of a tall building. Occupant discomfort due to wind-induced motion is strongly dependent on the turbulent and buffeting characteristics of the wind and presently there exists no satisfactory computational procedure to determine these effects (McNamara et al. 1997). Wind tunnels are generally utilized to determine estimates of acceleration response levels. The dynamic characteristics of the structure are calculated (vibration period, mass) and an estimate of the natural damping is made based on the type of the lateral load resisting system and the materials used in construction.

Predictions of acceleration response levels for various assumed damping values can be generated by wind engineers. A level of damping is then selected which satisfies the appropriate design criteria.

The parametric adjustments in the structural properties; mass, stiffness, and damping can change the overall structural response. The acceleration response of a building is given by the dynamic equation of motion

$$Acceleration = \frac{(wind\ force) - (Elastic\ force) - (Damping\ Force)}{Mass} \tag{3.3}$$

From Eq. (3.3), when the stiffness is increased to reduce acceleration, the ratio of stiffness to mass ratio governing the natural frequency of building) should be considered in the structural design. Slope of the load spectrum with frequency should also be taken into account.

Addition of structural damping or energy dissipation devices, such as passive energy absorbers and active feedback control systems to the basic structural system of buildings, results in the reduction of acceleration.

3.16 Damping Systems

Kareem & Tamura (1994) have recently summarized the various types of systems currently being used throughout the world. Damping systems can be categorized as passive semi-active and active. The passive systems may be further categorized based on their mechanism of energy dissipation, i.e., mass or inertia effects and direct energy absorption systems. More details of common damping systems are reviewed and discussed below.

3.16.1 Passive Damping Systems

Passive damping systems for the control of building motions are generally divided into the following categories: 1) direct energy dissipating systems and 2) inertia systems.

Most direct energy dissipating systems reduce the dynamic response by utilizing the relative displacement between the adjacent floors or other components of a building. These methods imply that the system must be capable of dissipating large amounts of energy through the movement of relatively small inter-storey displacements since the performance is related to small (often recurring) wind storms.

On the other hand, inertia systems are activated by the total dynamic motion, usually a translational displacement near the top of the building. Inertia systems can be conceptually thought of as inputting a force (whose magnitude depends on the inertia mass specified) at the top of the building operating 180°out of phase with the building response. One of the major disadvantages of inertia type systems is that they usually require a significant amount of valuable floor space near the top of the structure. Early applications of the tuned mass damper concept (Citicorp Center, New York and Hancock Tower, Boston) utilized heavy concrete or lead masses. More recent applications have utilized the tuned liquid column damper concept to provide multiple mass dampers and to provide broad-band response reductions.

Among the systems that impart indirect damping through modification of the structural system, the most popular concept is the damped secondary inertia system, e.g., tuned mass dampers (TMDs) and tuned liquid dampers (TLDs) including tuned sloshing (TSDs) and tuned liquid column oscillation (TLCDs) type dampers (Kareem (1992); McNamara & J (1977); Fujino et al. (1992; Sakai, et al., 1989). TMDs are activated by the total dynamic motion near the top of the building.

Tuned mass damper (TMD) is a device consisting of a mass attached to a building via a spring-dashpot system and vibration energy is dissipated by the dashpot when a relative motion develops between the mass and the building. TMDs can be conceptually regarded as inputting a force (whose magnitude depends on the inertia mass specified) at the top of the building operating out of phase with the building response. Only in recent years large scale tuned mass dampers are being used to reduce wind-induced vibration of tall buildings and structures such as the Centrepoint Tower, Sydney; the CN Tower, Toronto; the John Hancock Tower, Boston, and the Citicorp Center, New York.

Like a TMD, the TSD and TLCD impart indirect damping to the system by modifying the frequency response of the system; thereby reducing structural response. The damping systems that involve direct dissipation of energy include viscoelastic dampers, friction, lead and impact type dampers. The viscoelastic dampers are the most promising in this class (Keel & Mahmoodi 1986).

Vickery, Davenport & Wargon (1970) as well as Isyumov (1994) conducted aeroelastic model tests of a proposed structure and a tall building, and also carried out parametric studies of tuned mass dampers based on a two-degree-of-freedom system model and white noise excitation. McNamara & J (1977) and Luft (1979) discussed the design of large-scale tuned mass dampers. Tanaka & Mak (1983) chose a 1:100 scaled aeroelastic model of the CAARC Standard Tall Building as the wind tunnel test model, and considered wind excitation as band-limited white noise excitation to conduct parametric studies of the tuned mass damper.

Kwok (1984) performed full-scale measurements of wind-induced accelerations of the Sydney Centrepoint Tower, which has two passive tuned mass dampers, one near the top and one about half way up the tower. Xu, Samali & Kwok (1992) performed the parametric study of passive tuned mass dampers. Their study, leading to theoretical analysis and design of an effective and efficient tuned mass damper system, was based on excitation spectra which were directly measured from the wind-tunnel model tests. Theoretical results were in good agreement with the test results. All these studies have indicated that TMD systems are effective for reduction of wind-induced dynamic response of tall buildings and structures.

The TLDs are finding increasing popularity in Japan. The Gold Tower is equipped with a rectangular liquid tank with nets to enhance damping. Tokyo International Airport Tower at Haneda utilizes a TLD with floating particles to enhance damping in part due to surface tension. Shin Yokohama Prince Hotel is equipped with shallow circular containers at the top floor of the hotel. The full-scale tests have shown that the system is effective in reducing response for improving serviceability, and also reduces response in the range of 50-70% at wind speeds above 20 m/sec (Kareem & Tamura 1994).

Viscoelastic dampers utilizing hysteretic behaviour have been used in the World Trade Centre, New York, Columbia SeaFirst Building, Seattle, Two Union Buildings, Seattle, and Chientan Railroad Station Roof, Taipei (Nielsen et al. 1996). Shibaura Seavans Building and Chiba City Gymnasium in Japan also used viscoelastic dampers (Kareem & Tamura 1994).

3.17 Variable Damping Devices

Variable damping devices reduce the structural responses through changing the damping properties of the controlled structure based on the feedback signal and the control algorithm. These damping devices include the variable-orifice dampers, the variable friction dampers, the controllable-fluid dampers and so forth.

The variable-orifice damper, as illustrated in Figure 3.6, is realized through using a controllable, electromechanical, variable-orifice control valve to alter the resistance to flow of a conventional hydraulic fluid damper. Feng and Shinozuka (1990) first proposed the concept of applying the variable-orifice dampers to control the motion of bridges under seismic excitations (Feng 1990). Subsequently, this kind of dampers were studied analytically and experimentally by a number of researchers including Kawashima and Unjoh (Kawashima 1994). Sack and Patten (Sack 1993), Patten et al. (Patten 1996), Symans and Constantinou (Symans 1999), Nagarajaiah (Nagaragaiah 1994), and Yang et al. (Yang et al. 1995). Of these research efforts Sack and Patten (Sack 1993) implemented experimental study in which a hydraulic actuator with a controllable orifice was installed in a singlelane model bridge to dissipate the energy induced by vehicle traffic. This full-scale experiment on interstate highway I-35 in Oklahoma constitutes the first full-scale implementation of structural control in the United States. Findings in these research indicate that the variable-orifice damper is effective in reducing the structural responses due to external excitations.

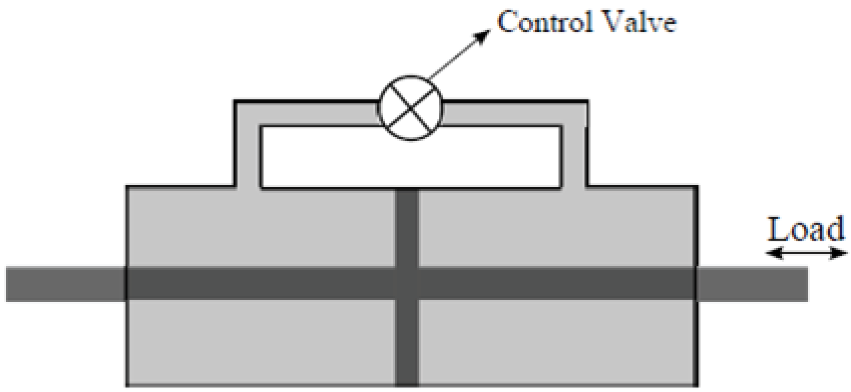


Figure 3.6 Schematic model of a variable-orifice damper

3.17.1 Variable-friction dampers

Variable-friction dampers dissipate kinetic energy in a structural system by means of utilizing forces resulting from surface friction. Akbay and Aktan (Akbay 1990, Aktan 1991) proposed a variable-friction device, which consists of a friction shaft that is rigidly connected to the structural bracing.

Kannan et al. (Kannan et al. 1995) developed a full-scale prototype device called the Active Slip Bracing Device (ASBD) which uses Coulomb friction. Energy is dissipated when the actual load exceeds the axial strength of the dissipater. The active characteristics of the device were implemented by a computer controlled clamping mechanism on the friction interface. The effectiveness and the feasibility of the ASBD were demonstrated by the result. Feng et al. (Feng et al. 1993) used the semi-active friction-controllable fluid bearing in parallel with a seismic isolation system. The variable friction is realized by means of changing the pressure in the fluid chamber of a bearing. It was illustrated that the variable friction force made the isolation system more effective in controlling the structural responses under earthquakes with a broad range of intensity. Yang et al. (Yang et al. 1994), Hayen et al. (Hayen et al. 1994) used active interface friction to control the response of structural systems. In addition, Yang et al. (Yang et al. 2002) studied the variable-friction dampers for reducing seismic responses of nonlinear buildings against near-field earthquakes. The authors used the combination of the base isolation system and the semi-active friction damper. It was found that the combination of the isolation system and the variable friction system is quite effective in preserving the integrity of buildings subjected to near-field earthquakes.

3.17.2 Controllable-fluid dampers

Differing from the aforementioned two kinds of variable damping devices, the third kind of variable damping device uses controllable fluid (Figure 3.7). This damping device is more reliable than the other two dampers because it contains no moving parts other than the piston. Controllable-fluid damper is a semi-active control device which uses controllable fluid in a fixed-orifice damper. Two fluids used to control the characteristics of dampers are: (1) electrorheological (ER) fluids and (2) magnetorheological (MR) fluids. A number of ER dampers were developed, modelled and tested for civil engineering structures (Ehrgott 1992, Gavin 1994, Gordaninejad 1994) where the effectiveness of the ER dampers were observed. However, the MR dampers have been demonstrated to be superior to its ER counterpart in the following aspects. It is found by researchers (Spencer 1997) that MR damper is tractable because the transition of rheological equilibrium can be achieved in a few milliseconds. Carlson et al. (Carlson et al. 1994) showed that the MR damper has higher yielding stress and is more robust against the variation of temperature (−40◦C -150◦C) in comparison with ER dampers. In addition, MR dampers can be easily controlled by low power (0-50 W), low voltage (12-24 V) and low current (1-2 A). Hence, MR dampers are widely used to control structural seismic responses. Therefore, MR dampers have been an attractive alternative to ER dampers.

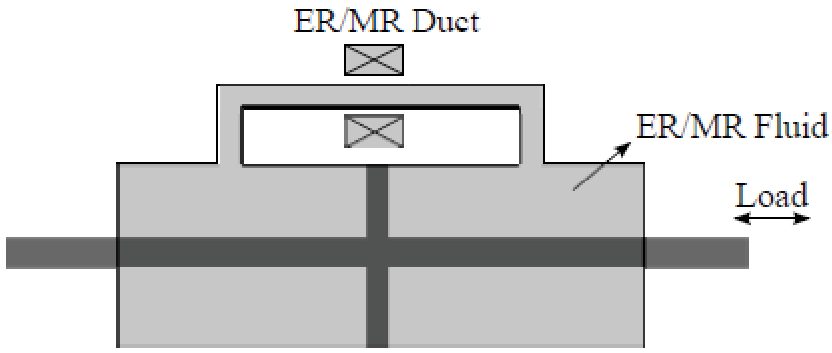


Figure 3.7Schematic model of a controllable-fluid damper

3.18 Variale Stiffness Devices

Kobori et al. (Kobori et al. 1993) proposed the first active variable-stiffness (AVS) system realized through a variable-orifice damper using on-off mode, to investigate semiactive control of the Kajima Research Institute building. Characteristics of the target building, such as the stiffness, can be adjusted during an earthquake such that the building is maintained in a non-resonant state. It was found in that the integrity of the building can be preserved through using a relatively small amount of energy under moderate or severe earthquakes. Based on a similar AVS system and the sliding mode control theory, Yang et al. (Yang et al. 1996) proposed control methods. The authors demonstrated numerically that the AVS system controlled by the proposed algorithm can suppress the seismic responses effectively. However, the controlled stiffness value of the AVS is switched between a zero and a fixed non zero value, which has potential issues of abruptness during the variation of different stiffness states.

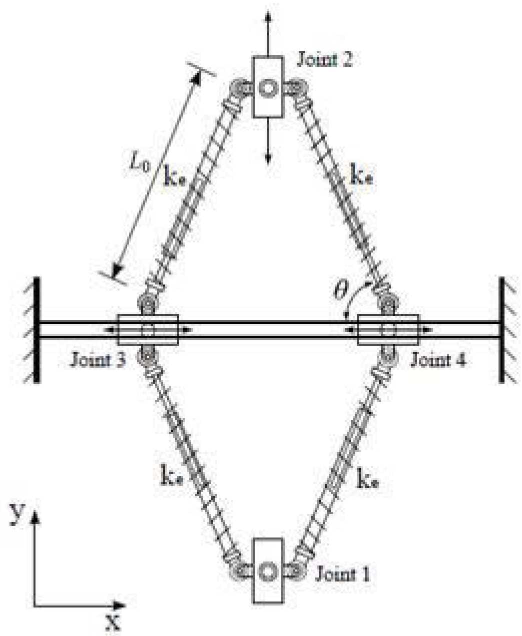


Figure 3.8 Semi-Active and Independently Variable Stiffness (SAIVS) device and STMD [2]

In order to address the potential problems of abruptness in the AVS system, Nagarajaiah (Nagarajaiah 1998, 2000) developed a semi-active continuously and independently variable stiffness (SAIVS) device as shown in Figure 3.8, and the ability of the device to vary its stiffness smoothly and reliably was demonstrated. The author and his co-workers utilized the SAIVS device in isolation system and semi active tuned mass damper. Nagarajaiah et al. (Nagarajaiah et al. 2005, 2006, 2007) studied smart base-isolated structures with SAIVS device and MR dampers. Their findings indicated that significant response reduction can be achieved by the combination of SAIVS and MR dampers. In addition, the combination of SAIVS device and MR dampers was employed in smart tuned mass dampers by Nagarajaiah and his co-workers (Nagarajaiah et al. 2005, 2007, 2013). The smart tuned mass damper with variable frequency and damping ratio was shown effective in reducing the seismic and wind induced vibrations.

In addition to the aforementioned AVS and SAIVS device, Nagarajaiah et al. (Nagarajaiah et al. 2013)proposed a new adaptive passive stiffness (APS) device which produces adaptive stiffness in a novel adaptive passive manner. The APS device is designed, modelled and tested experimentally.

3.19 Traditional Linear Tuned Mass Damper

A traditional linear Tuned Mass Damper (TMD) consists of a mass, a linear spring and a damper as shown in Figure 3.9. It is attached to the primary structure to transfer and dissipate the kinetic energy, thereby protecting the primary structure from excessive vibrations. When its frequency and damping ratio are appropriately tuned, the TMD can effectively dissipate the transferred energy in the neighbourhood of the target resonant frequency.

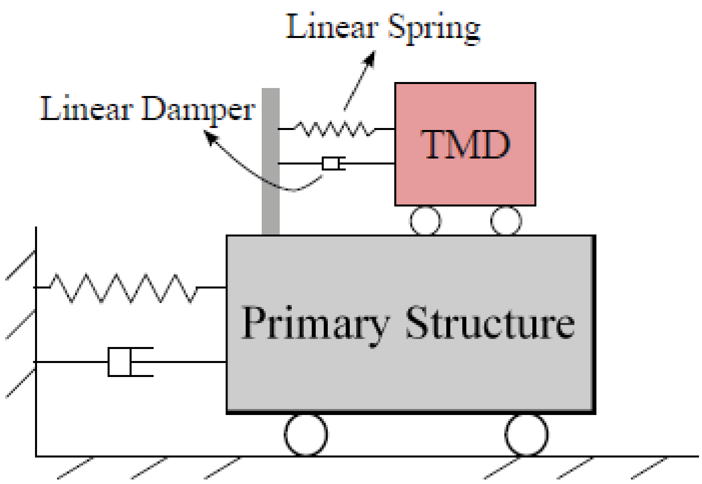


Figure 3.9 Illustration of a schematic model of a TMD

Since the Tuned Mass Damper (TMD) was first proposed by Watts (Watts 1883) and later patented by Frahm (Frahm 1909), it has attracted intensive interest and attention from the researchers and engineers in the community of vibration control. The first analytical contribution on the TMD is attributed to Ormondroyd and Den Hartog who analyzed the behaviour of undamped and damped TMD in undamped main system (Hartog 1928). Den Hartog then presented the design formula for the optimal TMD (Hartog 1956).The proposed optimal design is appropriate for undamped primary system subjected to harmonic loadings. Ioi et al. (Ioi et al. 1978) then established correction factors of these optimum absorber parameters in the case of light damping in the main system using empirical formulas. After that, Warburton and Ayorinde (Warburton et al. 1980,1982) further studied the optimum design of the TMD for damped main system under different optimal objectives and various excitations. The optimum design formulas according to each of the optimal objectives and excitations were summarized and tabulated in (Warburton 1982).

A well-established fact is that the optimum design of a TMD depends on the excitation and the structural response to be minimized. While seismic loadings are stochastic and non-stationary in nature, the optimum design formulas proposed by different researchers (Hartog 1956, Ioi 1978, Warburton 1980 and 1982), which is suitable for harmonic loadings, might lose their effectiveness under seismic excitations. In order to extend the effectiveness of TMD to seismic protection, a series of research (Wirsching 1973, Sladek 1983, Villaverde 1985, 1993 and 1995) were conducted to explore the optimum design under earthquakes.

Gupta and Chandrasekaren (Gupta et al. 1969) investigated the reduction effect of TMDs with elastic-plastic properties for a SDOF structure subjected to the S21W component of the Taft acceleration. Their findings indicated that TMDs are not as effective in reducing structural responses under seismic excitations as they are in reducing responses under harmonic excitations. Kaynia et al. (Kaynia et al. 1981) evaluated the effectiveness of TMDs for reducing fundamental mode response by using an ensemble of 48 earthquake accelerograms. They found that TMDs are less effective than expected. Sladek and Klingner (Sladek et al. 1983) analyzed a TMD designed using Den Hartog (Hartog 1956) formula. The TMD was placed on the top floor of a 25-storey building subjected to the S00E component of Elcentro accelerogram, Imperial Valley earthquake. Similarly, the authors concluded that the TMD is ineffective.

In comparison, Wirsching and Yao (Wirsching et al, 1973) studied the first mode response of a five- and ten-storey building with 2% damping ratio and subjected to a non-stationary ground acceleration. They tuned the TMD’s frequency to the fundamental frequency of the structure, and used a 20% damping ratio. Considerable reduction of response was observed. In addition, optimized the selection of TMD parameters and they achieved effective reduction effect. Afterwards, Villaverde et al. (Villaverde et al.1985, 1993, 1994, 1995) investigated the effect of TMD parameters for seismic application. Their results indicated that the TMD should be in resonance with the main structure and a design formula for the damping ratio of the TMD was proposed (Villaverde 1985) for effective seismic response reduction.

Based on the conclusions presented by Villaverde et al. (Villaverde et al.1985, 1993, 1995), (Sadek et al. 1997) numerically obtained the optimal tuning frequency ratio and damping ratio for different mass ratios of the TMD and damping ratios of the primary structure. Through regression with respect to the computed optimal data, the authors proposed an optimum design formula, which is a function of the mass ratio of the TMD and the damping ratio of the primary structure, for the TMD under seismic excitations. The authors examined the design formula using around 50 representative recorded seismic accelerations, showing that the proposed optimum design formula for the TMD is effective in reducing structural responses under seismic excitations.

3.20 Tuned Liquid Damper (TLD)

Another idea to reduce the vibration of the primary structure is to use the tuned liquid dampers (TLDs), as shown in Figure 3.10, where liquids serves similar purposes as the mass block does in a TMD. In this case, the liquid provides not only the secondary mass, but also the damping through viscous action primarily in the boundary layers (Soong 1997). The benefit of using TLD includes low cost, easy installation, non-constraints to the unidirectional excitation, few maintenance requirements and so forth (Fujino 1992). The idea of using a TLD to reduce vibrations in civil engineering structures began in the mid-1980s. Bauer (Bauer 1984) firstly suggested using a rectangular container filled with two immiscible fluids to suppress response through the motion of the interface. Welt and Modi (Welt et al. 1989) used a TLD in buildings to reduce overall response during strong winds and earthquakes.

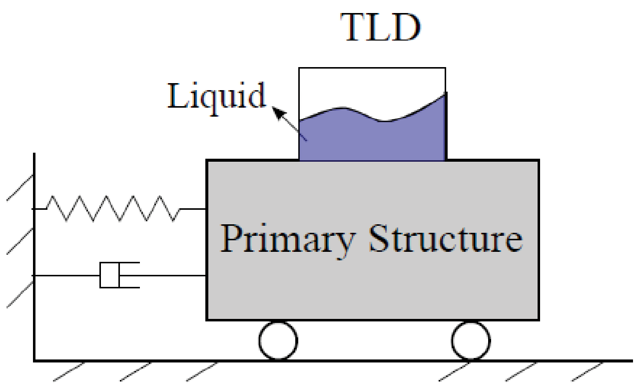


Figure 3.10 Illustration of a schematic model of a TLD

However, unlike the TMDs which are purely linear, the TLDs have inherent nonlinearity due to the slashing of the liquids and the presence of orifices. In order to understand and quantify the behavior of the TLDs, considerable research effort has been spent on the modeling of the TLDs. Early work done by Housner (Housner 1963) considered only linearized response. Then Miles (Miles 1985), Shimizu and Hayama (Shimizu 1987), Lepelletier and Raichlen (Lepelletier et al. 1988), and Fujino and Sun et al. (Fujino and Sun et al. 1992) further investigated the behavior of the liquids in the container and examined the effectiveness of the TLDs. These research efforts illustrated the characteristics of the TLD, providing easy-operated criteria for real application. Applications of the TLDs took place firstly in Japan. Examples of TLD-controlled structures are the Nagasaki Airport Tower installed in 1987, the Yokohama Marine Tower also installed in 1987, the Tokyo air traffic control towers at haneda and narita Airports installed in 1993 and so forth. Although a linear TMD or a TLD is effective in attenuating vibration at a specific excitation frequency, perhaps the most significant limitation is its narrow effective bandwidth in the frequency domain.

When the natural frequency of the primary structure shifts due to structural degradation or other reasons, a linear TMD can act instead as a vibration amplifier, increasing the response amplitude of the primary structure. In order to address this issue, researchers proposed multiple TMDs (MTMDs), nonlinear TMD (NTMD), semi-active Tuned Mass Damper (STMD) and Active Tuned Mass Damper (ATMD) which will be illustrated in the following subsections.

3.21 Multiple Tuned Mass Dampers

In order to overcome the aforementioned limitations in a TMD, multiple Tuned Mass Dampers (MTMDs) were proposed, analyzed and tested. As its name suggests, MTMDs consists of a number of TMDs which are attached to the primary structure in parallel as illustrated in Figure 3.11 or in series as illustrated in Figure 3.12 Hereby the MTMDs in parallel are introduced first.

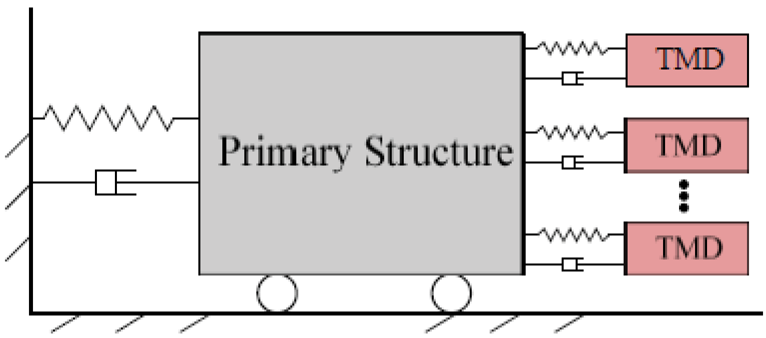


Figure 3.11 Schematic model of multiple TMD (MTMDs) in parallel

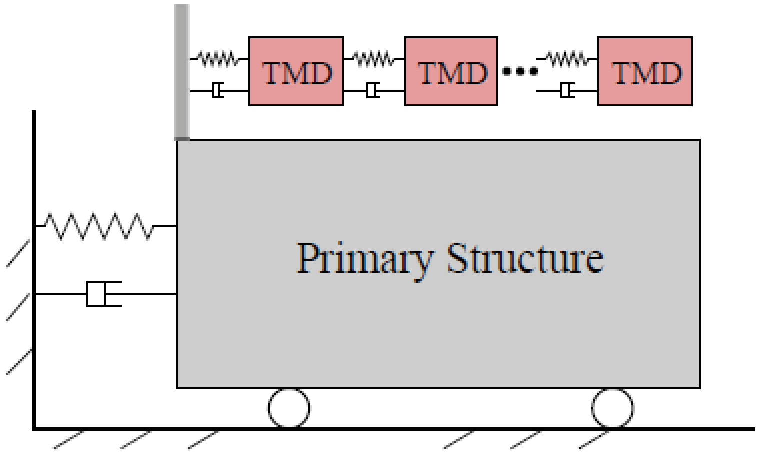


Figure 3.12 Schematic model of multiple TMD (MTMDs) in series

Iwanami and Seto (1984) [65] proposed dual Tuned Mass Dampers (2TMD) to attenuate the structural responses under harmonic excitations. The authors established optimum design for the 2TMD and demonstrated its better robustness than a single TMD. However, the improvement of the robustness does not seem to be significant enough.

Igusa and Xu (Igusa et al. 1990, 1991) first proposed the application of Multiple Tuned Mass Dampers (MTMDs) with distributed natural frequencies over a given frequency range. The dynamic system is subjected to random excitations. Yamaguchi et al. (Yamaguchi et al. 1993) the authors established the explicit formula for the impedance of the MTMD based on an asymptotic analysis method. It was found that the optimal design of the MTMDs has distributed frequencies centreed around the natural frequency of the primary structure. The MTMDs was demonstrated to be more effective and robust than a single TMD with equal total mass. Xu and Igusa (Igusa et al. 1992) further analyzed the effect of the substructures on the response of the primary structure where the substructures have equal stiffness and equally spaced natural frequencies. It was summarized that the behaviour of the multiple sub-oscillator have limited performance when the number of the sub-oscillators becomes large and the natural frequency becomes closely spaced. The behaviour of the multiple sub-oscillator can be represented by an equivalent damping when the natural frequencies span a sufficiently wide range. It was demonstrated that the effectiveness of the sub-oscillators is more significant than that of an equivalent single TMD when the damping of the primary structure is limited to low values.

Yamaguchi and Harnpornchai (Yamaguchi 1993) investigated the fundamental characteristics of the MTMDs under harmonic loadings. Analytical steady-state solutions for the primary structure and each of the MTMD are obtained. Effectiveness and robustness of the design parameters (frequency range, damping ratio, number of TMDs) were evaluated numerically based on the analytical solution. The results confirmed the advantage of a MTMD over a single TMD in reduction effectiveness and robustness. Furthermore, the authors revealed that there exists an optimum MTMD for the given total number of TMDs with the optimum frequency range and the optimum damping ratio.

Abe and Fujino (Abe et al. 1994) studied the modal properties of the MTMD-structure system and the effectiveness of the MTMD. Closed-form solutions for the modal properties (modal frequencies, damping and shapes) of the MTMD-structure system were derived using perturbation technique. Based on the closed-form solutions, the reduction effectiveness is examined through evaluating the equivalent damping added to the primary structure from the MTMD, revealing that an optimum damping ratio exists for the MTMD. The authors also proposed a critical bandwidth of the natural frequencies for the MTMD to make multiple tuning for the MTMD. Based on the overall results, a general design procedure regarding the mass ratio, the number of TMD, the damping ratio is summarized in the paper.

Abe and Igusa (Abe et al. 1995) further analyzed the performance of the MTMD used in structures with multiple vibration modes. Analytical results were obtained based on perturbation theory. It was illustrated that for structures with widely spaced natural frequencies, the response can be approximated by the response of the well-known single-mode structure/TMD system. In comparison, for structures with p closely spaced natural frequencies, at least p TMDs are needed to control the p closely spaced modes. In addition, the placement of the TMDs in the case of structures with p closely-spaced modes is demonstrated to be important. When the TMDs are placed inappropriately, their effectiveness will be limited. It was also found in the paper that the coupling of the closely spaced modes can be reduced by using certain TMD parameters and placements. The original system can be approximately represented by a set of decoupled SDOF structure/TMD systems.

Kareem and Kline (Kareem et al. 1995) evaluated the dynamic characteristics and effectiveness of the MTMDs under random excitations which were represented by wind and seismic loadings. Qualitatively similar findings were obtained as that concluded that under harmonic loadings. Furthermore, it was found that an optimum MTMD exists when the frequency range, total number of TMDs, damping ratio are selected optimally. It was also revealed that the MTMD with variable mass dampers or variable frequency spacing alone, or the combination thereof did not show any distinct advantage or disadvantage over uniformly distributed mass or frequency MTMD system. In addition, the frequency range was found to be the most important parameter in designing a MTMD, then comes the damping ratio and the number of MTMD. Li (Li 2000) conducted research on the effectiveness and robustness of a MTMD under harmonic ground acceleration. In the MTMD, the stiffness and the damping coefficient were fixed while the mass of each of the TMD were varied to obtain variable frequency and damping ratio. As a comparison, the MTMD with fixed mass and variable stiffness and damping coefficient is used and referred to as MTMD (II). It was found that the optimum frequency spacing of the MTMD is the same as that of the MTMD (II); the average damping ratio of the MTMD is a little larger than that of the MTMD (II). In addition, it was demonstrated that the optimum MTMD is more effective than the optimum MTMD (II) and a single optimum TMD. Additional research on the MTMD in a parallel form can be found in Refrences (Jangid 1995, 1999, Gu 2001, Chen 2002). Zuo (Zuo 2009) proposed another kind of multiple TMDs in series connected to the primary structure. Decentralized H2 and $H\infty$ control methods were used to optimize the parameters of spring stiffness and damping coefficients for random and harmonic vibration. It was found that the MTMD in series are more effective and robust than all the other types of TMDs of the same mass ratio. Differing from the optimum MTMD in parallel where the mass of each of the TMDs are identical or relatively close, the mass of the first TMD in the optimum MTMD in series is much larger than the others. In addition, the optimum MTMD in series has a zero damping ratio in one of its two connections.

3.22 Nonlinear Tuned Mass Dampers (NTMD)

It is well-known that the conventional TMD is sensitive to the structural variation such as damage, mass variation or other sources. The TMD will lose its effectiveness in reducing or will even amplify the structural responses when the frequency of the structure shifts. To address the limitations of the TMD, research effort on nonlinear Tuned Mass Damper (NTMD) with a nonlinear spring as illustrated in Figure 3.13 or a nonlinear damper was initiated since 1950s.

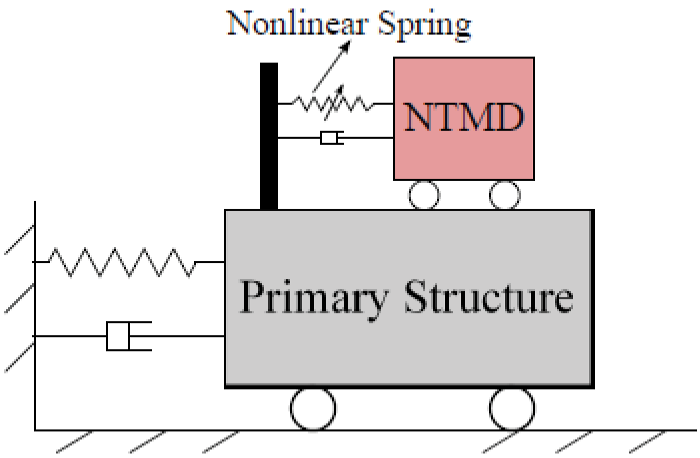


Figure 3.13 Schematic model of nonlinear TMD (NTMD)

Roberson (Roberson 1952) first investigated an undamped nonlinear dynamic vibration absorber consisting of a linear and a hardening cubic spring. He defined a 'suppression bandwidth' as a frequency band between the response peaks over which the normalized primary system response amplitude is less than unity. It was demonstrated that the suppression band for the nonlinear TMD was much wider than that of a TMD. This finding was later confirmed experimentally by Arnold (Arnold 1955). Roberson's work (Roberson 1952) was followed by Pipes (Pipes 1953) using a hyperbolic sine spring without damping. The author concluded that the nonlinearity in the spring can prevent the occurrence of sharp resonant peaks and to introduce odd harmonic components of relatively small amplitude in the motion of the absorber and primary system.

In order to further improve the performance of the NTMD, Snowdon (Snowdon 1960) studied the behaviour of a solid-type NTMD in reducing the primary structure response. It was demonstrated that an NTMD with a spring whose stiffness is proportional to frequency and a fixed damping factor can reduce the resonance of the primary structure considerably. The author later (Snowdon 1974) investigated alternatives for the spring-dashpot NTMD, such as a triple-element NTMD, indicating that if a third element is introduced in series, a 15% to 30% reduction can be obtained.

However, the promising reduction is quite sensitive to the tuning of frequency. Meanwhile, Masri (Masri 1972) considered the forced vibration of a family of piecewise linear two degrees of freedom (DOF) dissipative non-autonomous systems. Exact solutions were obtained and the asymptotic stability was confirmed. Analytical results showed that the properly designed NTMD combing features of dynamic neutralizers, Lanchester dampers, and impact dampers reduced some of the deficiencies inherent in the system, and was better than the conventional forms of TMDs. Hunt and Nissen (Hunt et al. 1982) introduced a viscous damper to an NTMD with a Belleville softening spring. Softening force-displacement curves were illustrated where the softening nonlinearity is controlled by the geometry of the Belleville washer. It was demonstrated that the effective bandwidth could be doubled compared to the case of a conventional linear TMD when a softening nonlinear spring is used. Use of such kinds of softening nonlinear spring will greatly increase the possibilities of the NTMD to reduce the unwanted vibrations to an acceptable level.

Kojima and Saito (Kojima et al. 1983) used an NTMD to attenuate the forced vibrations of a simply supported beam under sinusoidal excitation. The spring of the NTMD provides hardening cubic nonlinearity. Harmonic balance method was utilized to solve the D¨uffing equation which is reduced form of the original partial differential equation. In addition to the harmonic motion solution, the third-order super-harmonic and the one-third sub-harmonic were obtained. Based on the analytical results, a magnetic NTMD was optimally designed. Rice (Rice 1986) used a bow-type and shallow arch spring to provide the wanted nonlinearity for an NTMD. Harmonic balance method was used to achieve the approximate solution of the two DOF system. Based on the result, the author proposed the design guidelines for a nonlinear TMD working in narrow-band operating frequency. Soom and Lee (Soom et al. 1983) optimized the design of linear and nonlinear TMD using nonlinear programming techniques for a damped primary system. The authors examined the optimization criteria other than traditional ones. Small improvements were achieved in calculating the steady-state responses in the case of nonlinear springs.

Nissen et al. (Nissen et al. 1985), based on the design procedure proposed in Hunt's work (Hunt et al. 1982),realized the optimal design for a softening NTMD with a Belleville spring from a technical perspective, aiming to maximize the effective bandwidth. Afterwards, Jordanov et al. (Jordanov et al. 1989) proposed a numerical method for optimal design for linear and a nonlinear TMD in undamped and damped primary systems. The method of sounding was utilized to examine the objective functions and to find the optimal solution under multi-criteria. The effectiveness of the proposed algorithm for searching the optimal design was demonstrated in the research. Natsiavas (Natsiavas 1992) investigated the steady-state solutions and the stability characteristics of a nonlinear system consisting of a nonlinear primary structure and a weakly nonlinear TMD. An averaging method was used to obtain the approximate steady-state solution and Eigen analysis was performed to characterize the stability of the located oscillations. Two types of stability were encountered: one is the saddlenode type bifurcation and the other is the Hopf bifurcation. Based on the obtained steady-state solution, a parametric study was implemented for three cases: linear primary structure plus nonlinear TMD, nonlinear primary structure with linear TMD, nonlinear primary structure with nonlinear TMD. In each case, the representative result was illustrated. It was indicated that the selection of proper parameters of the nonlinear TMD would result in substantial improvements and avoid potentially dangerous effects.

Additional studies with respect to the behaviour of the nonlinear TMD (Vakakis 1999, 2003, Gourdon 2006, Nucera 2008) have demonstrated that a nonlinear TMD requires a much smaller mass than a linear TMD to achieve identical reduction effects and is capable of attenuating the transient oscillations of a main structure more effectively. However, response dynamics can become more complex any time nonlinearity is introduced. On one hand the NTMD has been demonstrated to be more effective than its linear counterpart as reported in the aforementioned literatures. On the other hand, potential problems such as instability and chaos will result due to the nonlinearity or the NTMD. Rice and McCraith (Rice et al. 1986) compared the steady-state solution of the linear primary structure coupled with an NTMD obtained from simulation and from harmonic balance perturbation method. Their results indicated that there was a possibility of combinational instability of the harmonic response in the suppression region if damping was kept low. When this instability occurs, the response of the primary system exhibits an almost-periodic oscillation with high amplitude and thus defeats the purpose of the vibration absorber. Shaw et al. (Shaw 1989) used the multiple scale method to study the steady state solutions of an NTMD with weak damping. The authors found that a combinational resonance could occur, resulting in large amplitude almost-periodic vibrations. This motion deteriorates the effectiveness of the NTMD and can coexist with the desired low-amplitude periodic response, which leads to initial condition dependent dynamics.

Gendelman et al. (Gendelman et al. 2006) identified that a small NTMD coupled to a linear oscillator will exhibit a quasi-periodic response when subjected to harmonic excitation in the vicinity of the main resonance of the system. Starosvetsky et al. (Starosvetsky et al. 2008), based on an analytical approximation, demonstrated the presence of a detached resonance on the frequency-response curve of a two degrees of freedom linear system coupled with an NTMD. Alexander and Schilder (Alexander et al., 2009) discovered a family of detached resonances in the lower frequency region for low forcing amplitudes in a primary linear structure when the linear spring stiffness of the NTMD vanishes. Recently, Sun et al. (Sun et al. 2013) investigated the attenuation of a hardening D¨uffing system using an NTMD and a semi-active TMD. The authors identified high amplitude resonance which is detached from the main response curve in the lower frequency range when the amplitude is relatively small. These high amplitude detached resonances undermine the performance of an NTMD when used as a passive device.

3.23 Pendulum Tuned Mass Damper (PTMD)

In addition to the conventional TMDs, the pendulum TMD (PTMD) consisting of a cable and a mass suspended at the top part of a building has received popularity in the community of vibration control in recent years. The most notable and largest TMD ever constructed in real world is the pendulum TMD installed on top part of Taipei 101 as illustrated in Figure 3.14.

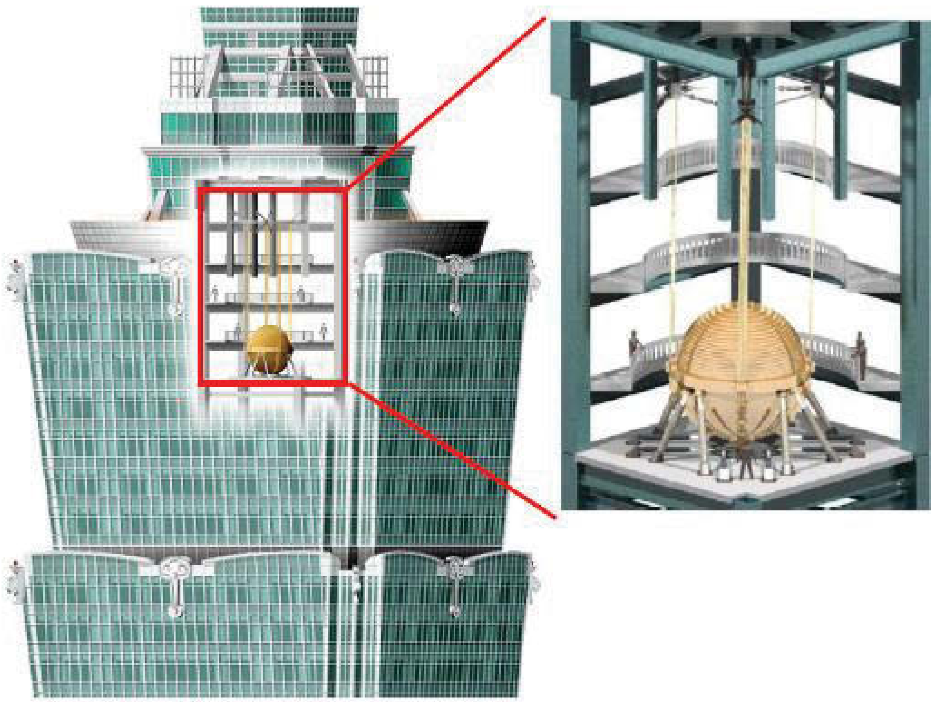


Figure 3.14 Illustration of the PTMD installed in Taipei 101. Adapted from sources

Mehdi et al. (Mehdi et al. 2006) used the PTMD to control excessive floor vibrations due to human movements. Their results indicated that a properly-tuned PTMD can effectively control the floor vibrations while an off-tuned PTMD may not function effectively. In order to overcome the off-tuning of PTMD, Nagaragaiah (Nagaragaiah 2009) further proposed a concept of adaptive-length pendulum TMD (APL-PTMD). It was shown experimentally that the APL-PTMD can significantly reduce the structural responses and outperforms its equivalent passive counterpart. Nagarajaiah, (Nagarajaiah 2009) the author also proposed the idea of using a rolling ball on a controllable guiding surface which is conceptually equivalent to a pendulum TMD but is easier to vary the radius. This idea was then confirmed by Matta et al. (Matta et al. 2009) where a rolling ball pendulum TMD moving on a three-dimensional guiding surface is studied theoretically and experimentally. The authors showed that this rolling ball pendulum TMD can reduce the structural responses in two mutually orthogonal horizontal directions.

Sun et al. (Sun et al. under review) further analyzed the performance of the STMD/APTMD under harmonic excitation and ground motion where closed-form solutions are derived. Their results provide physical interpretation with respect to how the design parameters influence the reduction effect of the STMD/APTMD. Based on the result presented by Sun et al. (Sun et al. under review) an optimal design for the STMD is proposed. In order to experimentally validate the result presented by Sun et al. (Sun et al. 2013), they (Sun et al. under review) used an adaptive pendulum TMD (APTMD) and an NTMD in parallel to attenuate the response of a D¨uffing system. Their results indicate that when an NTMD is used alone, a high amplitude detached resonance in the lower frequency range is identified. When the APTMD is used, the high amplitude detached resonance is greatly attenuated and significant attenuation of the structural responses over a large frequency range can be obtained. In addition, the APTMD can prevent the occurrence of the "jump phenomenon" existing in the nonlinear dynamic system. Of course, nonlinearity will be involved when the pendulum TMD experiences large displacement. As a matter of fact, a pendulum TMD is essentially a nonlinear TMD with softening nonlinearity(Bajaj 1994). In other words, the frequency response curve of a pendulum TMD, when large rotation happens, leans to the left [107]. In addition, those nonlinear characteristics, such as bifurcation, chaos within a nonlinear dynamic system can also result when a pendulum TMD is used. Research efforts on this aspect can be found in Lee et al. (Lee et al. 1999) and Song et al. (Song et al. 2003).

3.24 Semi-active Tuned Mass Damper (STMD)

When compared with the passive TMD, or TLD or NTMD, the semi-active TMD (STMD), controlled by a closed-loop algorithm, can adjust its damping, or frequency or both based on the feedback. Research effort has been focused on the subject since 1980s. Hrovat et al. (Hrovat et al. 1983) first investigated a semi-active Tuned Mass Damper (STMD) to control wind-induced vibrations in tall buildings, where the damping coefficient of the STMD was varied based on the feedback signal. The valve of the damper is controlled by an actuator connected to a computer which manipulated the motion of the valve based on the feedback and an optimal control algorithm. Linear Quadratic (LQ) control system design was used to obtain the control algorithm. Numerical simulations were conducted for three cases: an optimal TMD, an STMD, and a active TMD. It was indicated that an STMD could provide a reduction effect comparable to that of an active TMD and outperformed a conventional passive TMD. In addition, the energy consumption for the STMD is much smaller than that for an active TMD.

Abe and Igusa (Abe et al. 1996) proposed an STMD with non-zero initial displacement and time-varying damping to reduce the transient response of the primary structure. Based on perturbation method, approximate analytical expressions describing the transient response of the system were developed and the optimum initial displacement and the state-dependent damping were established. A small amount of energy is needed to set the initial displacement and state-dependent time-varying damping. Then the effectiveness of the proposed STMD and the associated optimum control method were examined in attenuating the transient response of a cantilever beam. Result indicated that the STMD was far more effective than a passive TMD. Meanwhile, Abe (Abe et al. 1996) evaluated the performance of the proposed STMD for seismic protection. Control algorithm was proposed and summarized in a flowchart to adjust the initial displacement and the damping. Numerical simulation was carried out under the El Centro recorded earthquake. It was found that the STMD and the control strategy was more effective than its passive counterpart.

In recent years, Nagarajaiah et al. (Nagarajaiah et al. 1998, 2000) developed a SAIVS device. The SAIVS can provide smoothly variable stiffness through adjusting the angle θ. Based on this, another type of Smart Tuned Mass Damper (STMD) have been proposed, modeled and tested. Nagarajaiah and Varadarajan (Nagarajaiah et al. 2005) studied the attenuation of wind induced vibrations of tall buildings using an SAIVS-STMD in which the variable frequency was realized through changing the stiffness of the SAIVS device. Short time Fourier transformation (STFT) was used to perform a time-frequency analysis with respect to the structural response, producing a time frequency representation of the response. Based on the result, a control algorithm was proposed for real time tuning of the frequency of the STMD. In order to demonstrate the effectiveness of the STMD, the result of a passive TMD and a active TMD (ATMD) from a benchmark example were also illustrated for comparison. It was indicated that the STMD is more robust and effective than the TMD in reducing the structural responses when there was stiffness uncertainty in the structure. The STMD could provide a comparable reduction to that of an ATMD, but with an order of magnitude less power consumption.

In order to examine the performance of the SAIVS STMD under some other excitations such as harmonic excitations, stationary excitations, and nonstationary excitations, Nagarajaiah and Sonmez (Nagarajaiah 2006) further used a single STMD and multiple STMD (MSTMD) to control the response of a multi-storey building. In this paper, a tracking of the excitation was carried out and analyzed using the STFT. A new semiactive control algorithm was developed based on the STFT result. It was found in the study that the STFT-based algorithm could track the frequency variation of the signal accurately and tune the frequency of the STMD effectively in real time. The STMD and the MSTMD were demonstrated to be the most effective when compared to their passive counterpart. In addition, when the structure suffers from damage, the TMD would become off-tuned while the STMD and the MSTMD were more robust. Recently, Sun et al. (Sun et al. 2013)studied the attenuation effect of a combined MTMD consisting of an STMD and an NTMD in parallel for a hardening D¨uffing system which is under harmonic excitation. Frequency of the excitation is tracked and assigned to be the natural frequency of the STMD. Parameter continuation method was used to compute the periodic solutions for the nonlinear system. Time integration method was also used to obtain the time-history of the system. Parametric study was conducted for various values of the design parameters in the system.

A high amplitude detached resonance curve could occur in the lower frequency range when an NTMD is used alone. However, this high amplitude detached resonance could be attenuated significantly using the STMD. In addition, it was found in the study that the combination of the STMD and the NTMD can effectively attenuation both the steady-state and transient responses.

Meanwhile, Eason et al. (Eason et al. 1995, 2013) used a combined MTMD consisting of an STMD and an NTMD in series to attenuate the response of a linear oscillator under harmonic excitations. It is demonstrated that the STMD is able to greatly reduce the amplitude of the primary structure response by acting to keep the nonlinear tuned mass damper within its linear range. Using a standard tuning approach for the STMD (modulating its stiffness to match the natural frequency of the STMD to that of the primary structure) and comparing its performance with an optimal passive linear tuned mass damper, the STMD provides a wider-band frequency-response amplitude reduction but often increases the response amplitude slightly above the resonance frequency.

3.25 Analytical Method for Analyzing Nonlinear Systems

Because the nonlinear dynamic system is very difficult to solve directly, researchers resorted to seek the approximate solutions. Perturbation methods, which are widely used in the field of nonlinear dynamics, are a technique in which the approximate solution is achieved in an asymptotic fashion. This approach can produce accurate result for weakly nonlinear structure experiencing relatively small oscillations. However, its accuracy will be weakened when strong nonlinearity and large displacement happens to the system.

3.25.1 Perturbation Method: Multiple Scales Method

The perturbation method is always used to achieve the periodic solution of nonlinear dynamic system. Several efficient methods that are frequently used includes the Av eraging Method, the Harmonic Balance Method, the Lindstedt-Poincare Method, the Multiple Scales Method and so forth (Nayfeh 1973). The procedure of computing the approximate solution using the Multiple Scales method is briefly introduced here. A general nonlinear dynamic system can be represented by:

$$F(x, \dot{x}; M) = 0 \tag{3.1}$$

Where x denotes the state-space vector; M is an m-dimensional parameter vector. Showing the time scales using a small parameter ε , i.e.

$$T_n = \varepsilon^n t \tag{3.2}$$

Then the derivative with respect to time t can be represented by these slower time scales as:

$$\cdot \frac{d}{dt} = \frac{dT_0}{dt}\frac{\partial}{\partial T_0} + \frac{dT_1}{dt}\frac{\partial}{\partial T_1} + \ldots = D_0 + \varepsilon D_1 \tag{3.3}$$

Assuming the solution x(t, ε)can be expanded to a series:

$$x(t,\varepsilon) = x_0(T_0, T_1, T_2, \ldots) + \varepsilon x_1(T_0, T_1, T_2, \ldots) + \varepsilon^2 x_2(T_0, T_1, T_2, \ldots) + \ldots \tag{3.4}$$

Substituting Eq. (3.4) and Eq. (3.3) into Eq. (3.1) produces:

$$F(x_1, x_2, x_3, \ldots, \varepsilon, \varepsilon^2, \varepsilon^3, D_1, D_2, D_3, \ldots) = 0 \tag{3.5}$$

Since the small parameter ε is arbitrary, the coefficients of the ε with different orders should equal zero to make the equation hold, i.e.

Then a set of equations are obtained. Solving these equations from low order term to high order term yields the expression of x_1, x_2, x_3....Eventually, the frequency response function which is a nonlinear algebraic equation can be obtained by means of eliminating the scalar terms. Then the frequency response curve can be calculated through numerically solving the nonlinear algebraic equation. Because the solution of the frequency response function is multi-valued of which the stability needs to be determined, which is discussed in the following subsection.

3.25.2 Local Stability Analysis

Stability of the solution needs to be determined through eigen analysis. The general Eq. 3.1 can be written as:

$$\dot{x} = G(t;M)x \tag{3.6}$$

Let $X_0(t)$ be the solution of the system at parameter M_0, i.e. $X_0(t) = G(t;M_0)$. Adding a small disturbance y(t) to $X_0(t)$, i.e.

$$x(t) = X_0(t) + y(t) \tag{3.7}$$

Substituting Eq. (3.7) into Eq. (3.6), expanding the result and retaining the linear terms in disturbance produces:

$$X_0(t) + \dot{y}(t) = G(X_0(t) + y(t);M) = G(t;M_0) + \left\{\frac{\partial G(t;M)}{\partial x}\right\} y(t) + O\left(\|y\|^2\right) \tag{3.8}$$

Eq. (3.8) reduces to:

$$\dot{y}(t) = A(t;M_0)y \tag{3.9}$$

Local stability of the solution X_0 can be determined be means of analyzing the eigen values of the matrix A. If A is a constant matrix, its eigen value and eigen vector can be calculated directly. Otherwise, Floquet theory is needed to determine the local stability also by means of analyzing the eigen value of a monodromy matrix (Nayfeh 1995). If the real part of all the eigen values are negative, the disturbance will eventually vanish and the examined solution is asymptotical stable. On the other hand, the solution is unstable if the real part of the eigen values are positive. In the case that the real part of the eigen value is zero, bifurcations will occur, which can be analyzed by keeping higher order terms when Taylor expanding Eq. (3.7). In addition, if the real eigen value change signs, a Saddle-node bifurcation might occur and if a pair of complex conjugate eigenvalues whose real part change signs, the Hopf bifurcation resulting in quasi-periodic oscillations might occur. It is noted that the rules listed here are general description of determining the local stability of the solution.

3.26 Numerical Methods For Analyzing Nonlinear Systems

In comparison with the analytical methods, numerical approaches are suitable for solving problems involving strong nonlinearity and large displacement. With the rapid progress of computers and high performance computing (HPC), numerical computation is playing a more and more important role in the field of science and technology. As for computing the solutions for nonlinear systems, two methods are always used: one is the Time Integration Method and the other is the parameter Continuation Method. The following subsections will introduce the principles of each of the two methods.

3.26.1 Time Integration Method

The most direct numerical method for computing the periodic solutions of a dynamic system is the so-called brute-force approach (Nayfeh 1995) which is essentially based on time integration. In this approach, the system is integrated at a given initial condition for a long enough time until the steady-state solution is reached. Advantage of this approach is that it is clear and easy to implement. Meanwhile, it is very general because it can calculate fixed points, periodic solutions, quasi-periodic solutions and chaotic solutions.

Mathematically, a dynamic system can be represented by a set of ordinary differential equations (ODEs) or partial differential equations (PDEs) together with the initial values (IV) of the boundary values (BV). Problems with initial values are referred to as IVP and the latter is BVP. Approximate numerical solutions for the IVP can be obtained through time integration. Generally, the numerical methods for IVP can fall into two large categories: explicit method and implicit method. Explicit method, as its name suggests, the value of the variables at the right hand side (R.H.S.) of an equation at each time step are known; hence, no iterations are needed to compute the value for the next step. In comparison, there are unknown variables at the R.H.S. when computing the value for the next time step and iterations are performed until the convergence criterion is satisfied. Generally, the implicit method has higher order accuracy and is more stable than the explicit method. Forward Euler method, the Runge-kutta method, and the Gaussian quadrature method belong to explicit method. Implicit method includes Back Euler method, the Adams-Moulton method(JohnC 2003), the well-known Newmark-β method which is widely used in structural engineering and so forth.

3.26.2 Continuation Method

Although the time-integration method is simple and general, it has several disadvantages:

(1) for light damped system, the convergence can be very slow because the transient response takes a long time to decay (2) not all the unstable solutions can be located by reversing the direction of integration and (3) it is difficult to judge whether the steady-state solution is achieved. In order to overcome these drawbacks, a more direct approach is proposed and used here.

In comparison with the time integration method, Continuation method for periodic solutions computes the solution through generating a continuum of periodic solutions with respect to a control parameter, say α, rather than direct time integration. The solution seeking procedure starts from an initial guess, say x_0, which is the solution corresponding to the starting point of the continuation parameter, say α_0. The initial guess can be obtained either analytically or numerically. Then the continuation parameter α is varied from α_0 to α_1 and the initial solution x_0 is updated to x_1 by means of solving a set of algebraic equations resulting from discretizing the original dynamic system. The procedure is similar to that for solving boundary value problems (BVPs). Arc length method which is widely used in BVPs is used in some software, like DERPER (Holodniok 1984) Another similar method Pseudo-Archlength method (Doedel 1986, 1991) which uses a pseudo-arc length constraint equation (arc length constraint equation is used in the Arc length continuation method) is used in AUTO (Doedel 1987) bifurcation and continuation software.

The principle of the Arc length method and the Pseudo-Arc length method are illustrated here:

Arc length Continuation

For a dynamic system defined by Eq. (3.1), let x(T(s),)(s),)(s)) with period T(s) be a periodic solution of the equation, where the arc length s is used as the continuation parameter and η denotes the state-space variable. Hence, the solution x(T(s), n(s), n(s)) satisfies the following equation:

G(T(s,f), the following equations,− η (s) = 0 (3.10)

with the initial condition: x(T(0),)(0),)(0)) = η (0). Differentiating Eq. (3.10) with respect to s yields:

$$\frac{\partial G}{\partial T}(T,\eta,\alpha)\dot{T}+\frac{\partial G}{\partial \eta}(T,\eta,\alpha)\dot{\eta}+\frac{\partial G}{\partial \alpha}(T,\eta,\alpha)\dot{\alpha}=0 \qquad (3.11)$$

where ($\dot{}$) denotes derivative with respect to s; ∂ den is a n×n matrix, ∂ atri is a n×1 matrix, ∂ maT is a n×1 matrix. Eq. (3.11) contains n linear algebraic equations while the system has (n+ 2) unknowns $(\dot{T},\dot{\eta},\dot{\alpha})$. Therefore, two additional equations are needed. One additional equation comes from the Euclidean arc length normalization:

$$\dot{\eta}^T\dot{\eta}+\dot{T}^2+\dot{\alpha}^2=1$$

(3.12)

Another equation is specified in the form of a phase condition, i.e. one variable η_k in the vector η along the continuation path is fixed.

$$\frac{d\eta_k}{d_s}=0 \qquad (3.13)$$

With the two additional equations, the (n+2) equations can be solved for the periodic solution x(T, equ).

Pseudo-Arc length Continuation

The principle of Pseudo-Arc length Continuation method differs from the Arclength method in that the two additional equations are specified in a similar but different manner. The first equation is specified using a phase condition in an integral form. Let s_0 and s be the two consecutive points on the branch; x_0 = x(T_0, $_0$, $_0$), $\hat{x}(t)$ are the two periodic solutions corresponding to the two points s_0 and s. If $\hat{x}$ is a solution, then $\hat{x}(t+\sigma)$ is also a solution for any σ. Then the phase condition is obtained when the distance $\|\hat{x} - x_0\|$ is minimized in regard to the time variation σ:

$$D(\sigma) = \int_0^T \|\hat{x}(t+\sigma) - x_0(t)\|^2 dt \tag{3.14}$$

Setting dD(σ)/dd = 0 produces.

$$\int_0^T \frac{\|\hat{x}(t+\sigma) - x_0(t)\|^2}{d\sigma} dt \tag{3.15}$$

Assuming the solution of Eq. (3.15) is σ*, i.e. $\hat{x}(t+\sigma^*)$ (3.14) reduces to:

$$\int_0^T [x(t) - x_0(t)]^T \dot{\imath} dt = \frac{1}{2} x^T x \Big|_0^T - \int x_0^T \dot{x} dt = -\int x_0^T \dot{x} dt \tag{3.16}$$

Integrating Eq. (3.16) by parts and using Eq. (3.1) produces:

$$\int_0^T x(t)^T - \dot{x}_0 dt = \int_0^T x_0^T x^T F(x(T_0, \eta_0, \alpha_0); \alpha_0) dt \tag{3.17}$$

The second equation is specified by the pseudo-arc length constraint:

$$\int_0^T [x(t) - x_0] \dot{x}_0 dt + (T - T_0)\dot{T}_0 + (\alpha - \alpha_0)\dot{\alpha}_0 = \delta_s \tag{3.18}$$

where $(\dot{})$ designates derivative with respect to the arc length s and δ s represents the step size along the continuation path. Eq. (3.11) together with Eq. (3.17) and Eq. (3.18) constitutes the (n+2) equations for the dynamic system with (n + 2) unknowns. Then the solution x(T(s), η (s), α (s)) can be achieved by means of solving the (n + 2) linear algebraic equations.

3.27 Summary

An overview with respect to the research effort focusing on several kinds of those widely used TMDs, including the conventional TMDs, the LTMDs, the NTMDs, the MTMDs and the STMDs, and the related variable damping and stiffness devices is presented in this chapter. The TMDs, which have been well understood and widely deployed in real engineering, have their limitations due to the narrow effective suppression bandwidth. In comparison, The MTMDs and the NTMDs can effectively broaden the suppression bandwidth. However, the application of NTMDs is more convenient than that of the MTMDs which needs a lot of effort in the process of design and installation. At the same time, it is demonstrated that the STMDs can provide comparable response reduction to that of the active TMDs yet they require an order of magnitude less power. Therefore, the NTMDs and the STMDs are the focus of this thesis. In addition, analytical and numerical methods used to solve nonlinear dynamic equations have been reviewed in this chapter. To sum up, the Continuation Method can be used to efficiently and accurately trace stable and unstable solution branches with respect to a predetermined control parameter. This method is especially useful in the analysis of nonlinear systems due to the complex and often unpredictable response behaviour. Additionally, the Psuedo-Arc length method provides the unique ability to trace folding solution branches. Therefore, in the current thesis, the bifurcation continuation software AUTO (Doedel 1997), which is based on the Psuedo-Arc length method, is used to compute the responses of the nonlinear dynamic system.

Chapter 4

4 Structural Response Under Wind Excitation 2D/3D Analysis

4.1 Mid-Rise (30 Storey) Structure - 2D Analysis

4.1.1 Introduction

To date the engineering community has seen structural façade systems as non-structural elements with high aesthetic value and a barrier between the outdoor and indoor environments. The role of facades in energy use in a building has also been recognized and the industry is also witnessing the emergence of many energy efficient façade systems. It has also been recognized that façade systems add some stiffness and damping to the overall building despite the new and modern systems, e.g., curtain walls, which add a relatively small amount of stiffness and damping to the overall building.

Despite these advancements, the façade has been rarely considered or designed as a potential wind-induced vibration absorber for tall buildings. In this chapter the potential of utilizing a moveable exterior façade in a double-skin façade is investigated and shown that with optimal choices of materials for stiffness and damping of brackets connecting the two skins, a substantial portion of wind-induced vibration energy can be dissipated which leads to avoiding expensive lateral stiffening systems and/or space consuming large damper systems such as tuned mass or liquid dampers.

Stochastic simulation of wind forces used in the finite element modelling have been detailed at the part 3.14.5.

The initial works have demonstrated that up to 50% reduction in accelerations and displacements caused by winds can be achieved by a smart and efficient façade design, including purely passive systems with constant stiffness and damping or better, by a smart system possessing variable stiffness for different phases of façade movement.

4.1.2 Structural Modelling

In order to evaluate the wind response of the proposed system, the main structure and the facade system are simplified and modelled as line elements using finite element method for analysis. The system consists of primary structure components (Beams, columns, and shear wall) representing the main building structure, and the façade panels simplified as a vertical line element.

The Figure 4.1 shows the floor plan and the major input data for the definition of the mid-rise test case buildings. Properties of façade system components and structural components of the studied building (i.e., Tables 4.1 – 4.4) have been addressed on page 74 of this research.

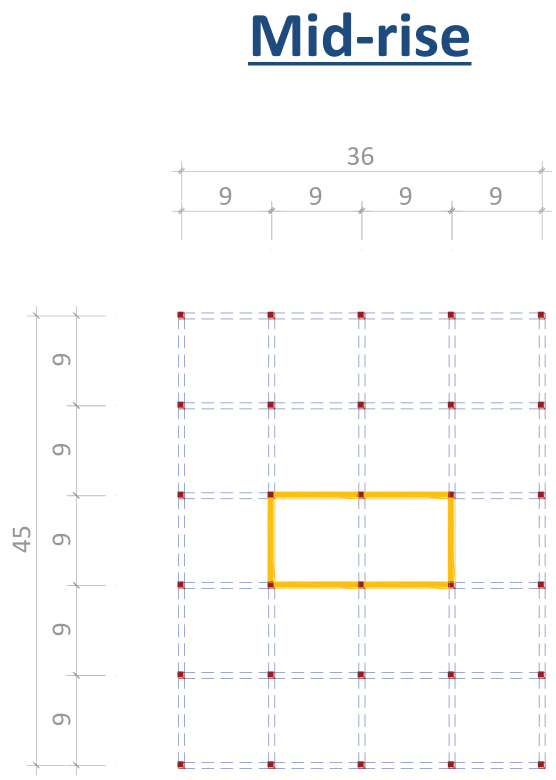


Figure 4.1 Mid-rise and high-rise test case building floor plant

At the first stage, shell elements are used to represent shear walls. As shown in Figure 4.1 shear walls are modelled in the mid bay to increase the lateral stiffness. The structure is designed based on Australia and New Zealand standard to obtain the shear wall section.

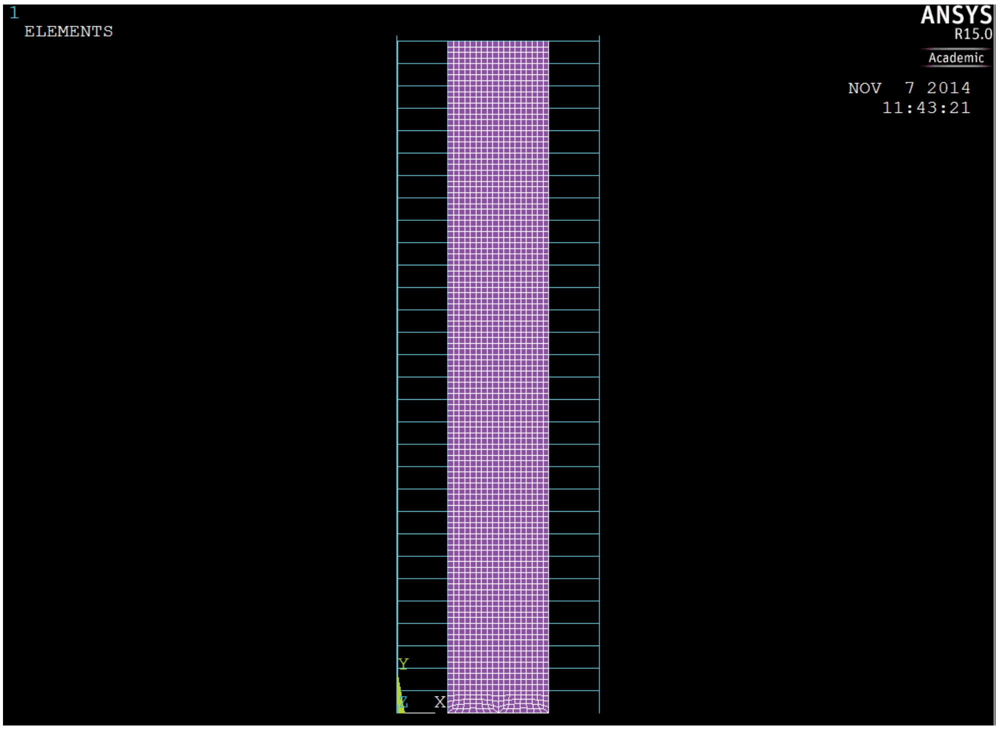


Figure 4.2 Finite element model of structure equipped with shear wall using shell elements

However, using shell elements is time consuming and high computing power is needed, especially with a multi linear façade damper behaviour which leads to the need for more simplified model. To obtain the same lateral stiffness diagonal bracing system is used to represent the behaviour of shear wall in the finite element modelling. It can be demonstrated that diagonal bracing can be a substitute system if the mass and the lateral stiffness of the diagonal bracing is similar to the shear wall system it is replacing. To carry out sensitivity analyses, static loads are applied to adjust the lateral stiffness. Modal analysis also used to adjust distributed mass and total stiffness of the system. Figure 4.2 represents the structure having similar mass and lateral stiffness behaviour.

We assume the mass of shear wall is equal to mass of brace however to get the lateral deflection of primary structure right, stiffness of braces is designed to meet the same lateral deflection.

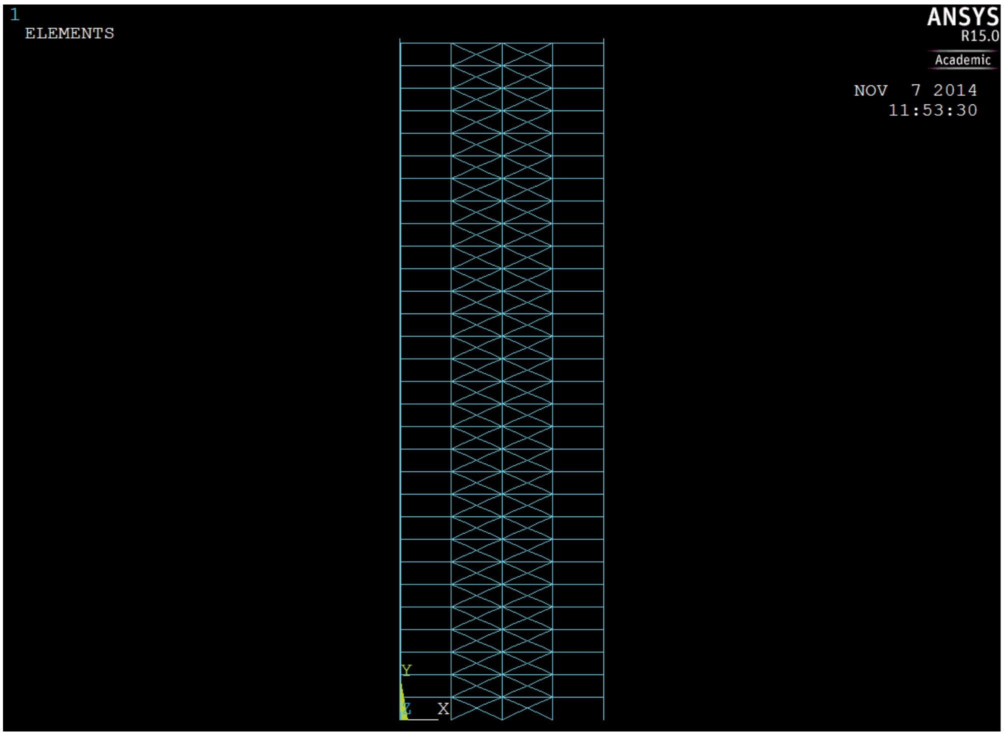


Figure 4.3 Finite element model of structure with shear wall using brace system

As stated earlier, static analysis is performed to match the lateral stiffness of shear walls and the simplified diagonal bracing system. Figures 4.3 and 4.4 show the maximum lateral displacement of two systems subjected to uniform static lateral load. It can be seen that both structures have similar maximum lateral displacements in X-direction. The vertical deformation is related to the total weight of the structure which is scaled to be presentable. The brace section and material density of braces are two variable parameters which should be adjusted based on static lateral load analysis and modal analysis to obtain similar behaviour.

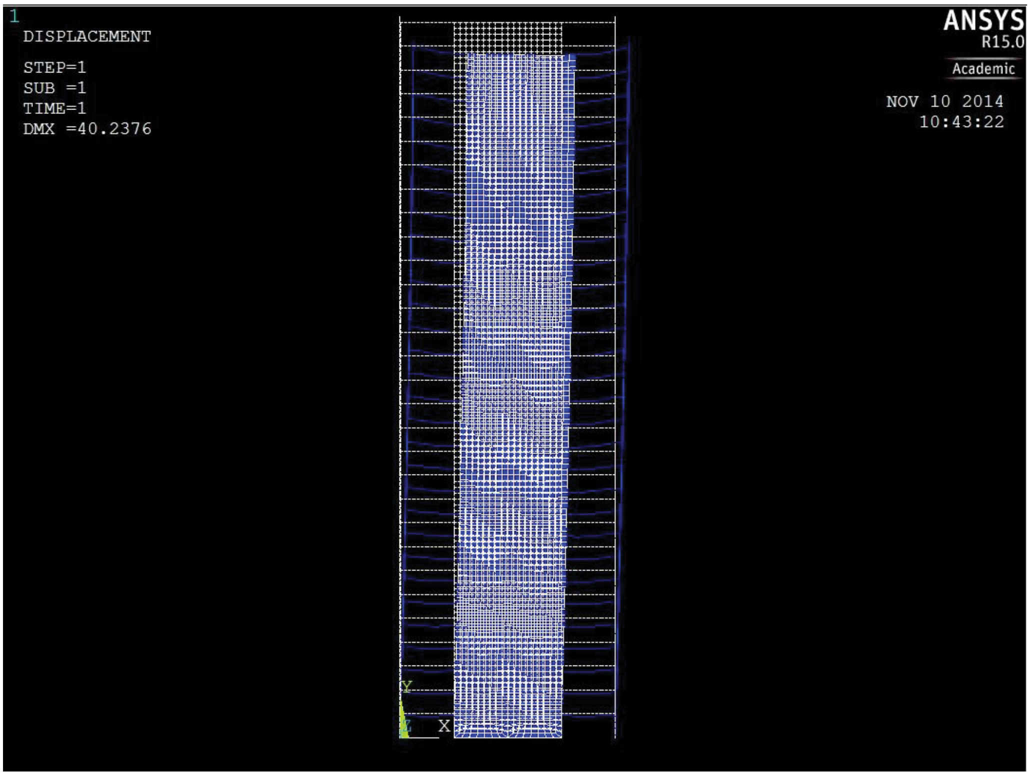


Figure 4.4 Deformed and undeformed structure equipped with shear wall subjected to linear static load

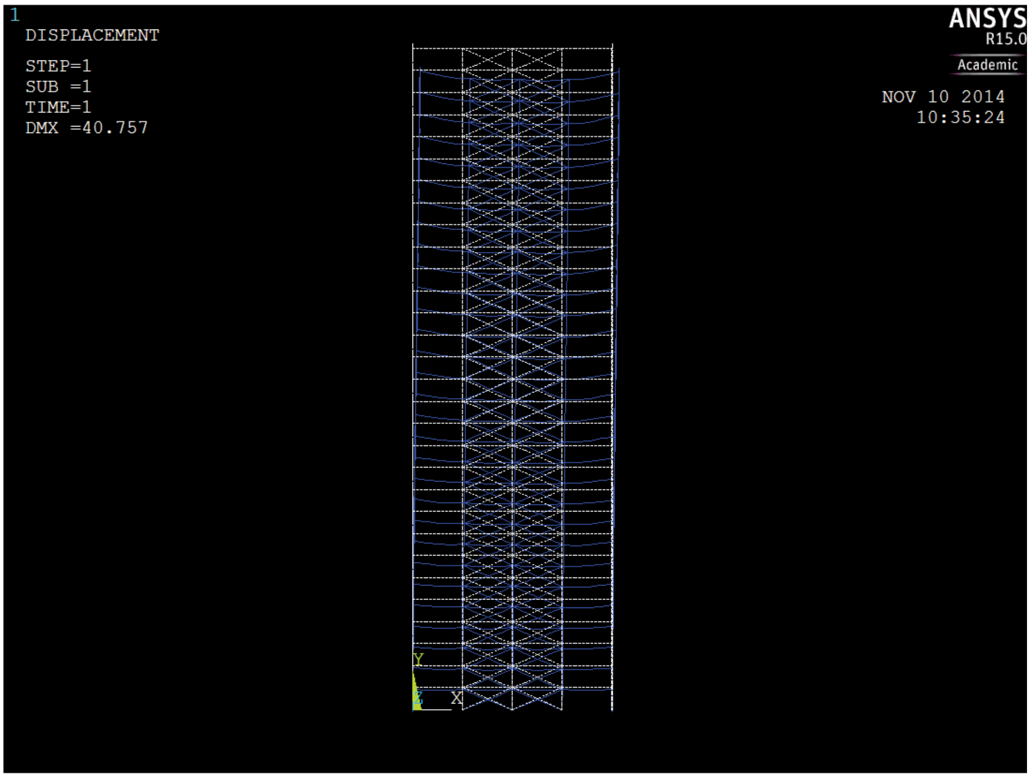


Figure 4.5 Deformed and undeformed structure equipped with a diagonal braking system subjected to linear static load

The façade system consists of vertical elements that represent the glass and aluminium frame together, which is a simplified model, and horizontal elements which represent the brackets to connect the façade system to the main structure. Figure 4.6 is presented in order to show façade panels and the connections that include dampers (works in X direction) and also a link (works in Y direction). Each façade panel is connected to the main structure using two types of elements. The link is used to carry the weight of each panel and the damper element that work in the in and out movement.

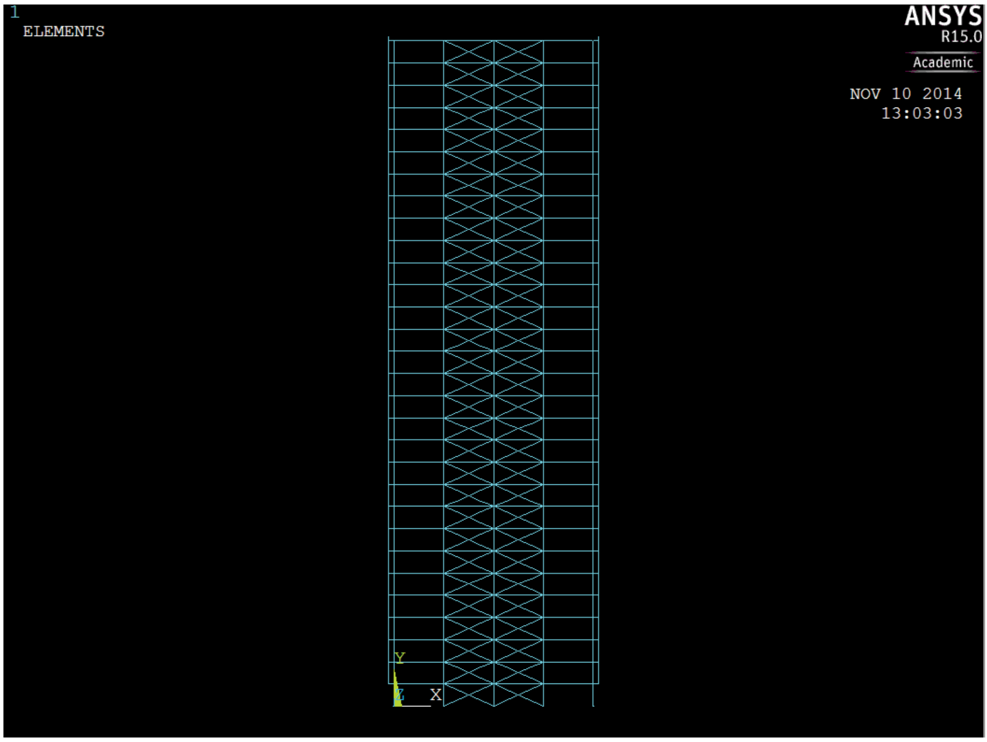


Figure 4.6 Finite element model of structure with façade system

Movable façade panels are attached and modelled on the top third of the structure; however, the remaining panels are connected rigidly to the main structure. Based on sensitivity analyses carried out at the beginning of the research, optimal/economical arrangement of smart panels were found to be at the top third of the structural height. One can of course use moveable panels for the entire height but this is too expensive to implement.

It can be noted that façade panels are designed to hang on brackets which leads to links to carry the weight of each panel. Finite element model in Figure 4.5 schematically shows the facade system without the first storey panels to confirm that those panels cannot actually transfer the weight to the ground.

Regarding the presented concept, façade panels in the top third of the structure move and due to right frequency movement, it can work like a filter as well as a multi tuned mass damper system to dissipate the wind energy. Figure 4.6 shows the 2D mid-rise structure when subjected to the wind loads. It can be seen that the façade in the top third is the structure moves back and forth in order to change the response of the main structure.

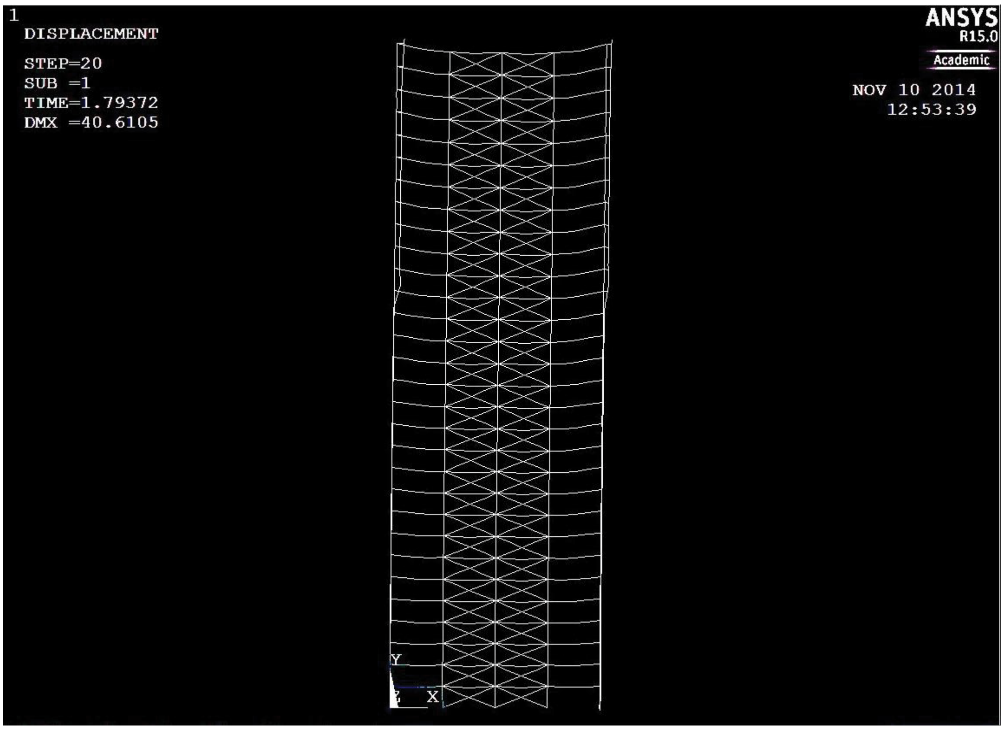


Figure 4.7 Response of structure the façade system subjected to wind load

Figure 4.7 illustrates the gap between the facade panel and primary structure, considered to be 200 mm, to provide enough space for damper installation but this can be changed based on the bracket design. As shown, the façade panels are attached to the primary building by some brackets shown as purple squares in Figure 4.7. These brackets are assumed to be flexible in order to absorb maximum external energy induced by the wind force and are considered as a non-linear damper element in ANSYS modelling. The small red circles in Figure 4.7 show the joints between adjacent panels.

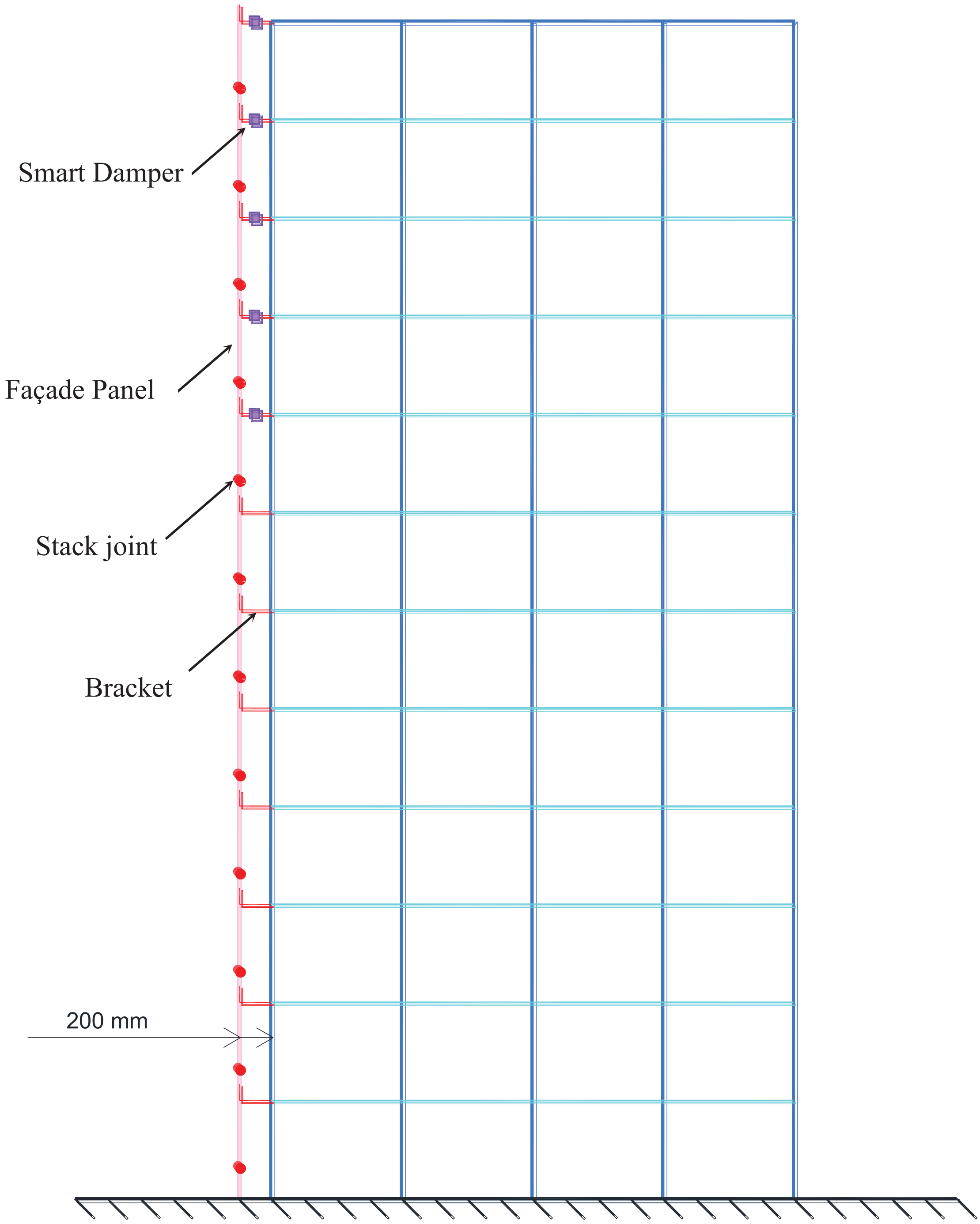


Figure 4.8 Schematic elevation view of the mid-rise structural model with movable façade on one side

Based on an optimisation process, movable façades were placed at the top one third of the structural height. The whole system hangs off each level with brackets which could move in the axial direction, but are fixed in the vertical direction. The joint between façade panels on each floor is assumed as a pinned joint and the panels are attached to each other from above and below by these pinned joints which can move freely during the application of environmental forces. The joint is located 900 mm above the floor slab level on each floor, as shown in Figure 4.8. Each panel is attached by a bracket to the flooring slab. The stack joint is designed to resist lateral loads while the two floor anchors resist gravity and lateral loads. One of the two floor anchors allows movement in a plane along with the unitized system.

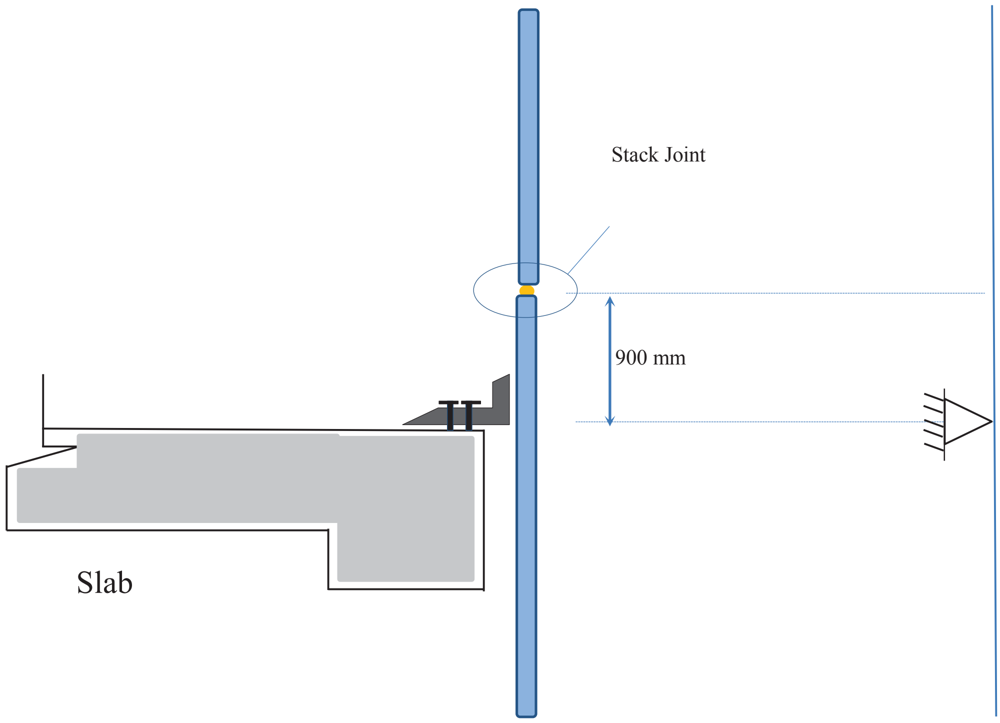


Figure 4.9 Detail of façade connection to the primary structure and modelling assumption in ANSYS

In table 4.1, modulus of elasticity, Poisson's ratio and density of materials used for façade components in ANSYS modelling are shown.

Table 4.1: Properties of façade system components

Façade Sections	Modulus of Elasticity (GPa)	Poisson's Ratio	Density (kN/m³)
Glass	62.76	0.2	22
Aluminium	68.64	0.33	27

Concrete material properties used in ANSYS modelling are presented in Tables 4.2 for mid-rise concrete structures. These values were obtained from common material properties used by Australian construction companies.

Table 4.2: Material properties of main mid-rise concrete structure

	Compressive Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio	Density (kN/m^3)
Column	60			
Beam	40	30	0.2	24.5
Slab	35			

The primary mid-rise concrete structure natural damping ratio is assumed to be 1% of critical, which is within the range of statistically reasonable values based on measured natural damping ratios for mid-rise buildings. As shown in Table 4.3, the bare frame has a frequency slightly less than the façade system with a movable façade. Concrete material properties of main structure are found in Table 4.4.

Table 4.3: Mid-rise structural model dynamic properties

Structure Type	Period (Sec)	Frequency (Hz)
Bare Frame	0.48	2.10
Frame with Movable facade	0.46	2.14

Table 4.4: Material properties of main mid-rise structure

	Compressive Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio	Density (kN/m^3)
Column	80			
Beam	60	30	0.2	24.5
Slab	50			

4.1.3 Façade Systems

The response of mid-rise buildings subjected to wind loads is dominated by their first natural frequency. Sensitivity analysis shows that working on other frequencies are not justified and using them to have the TMD effects could not lead to beneficial effects in terms of acceleration or displacement control of the main structure. Therefore, in order to have a significant TMD effect, the façade must vibrate at or near the first natural frequency of the building with enough energy (mass and displacement) as portrayed in Figure 4.9.

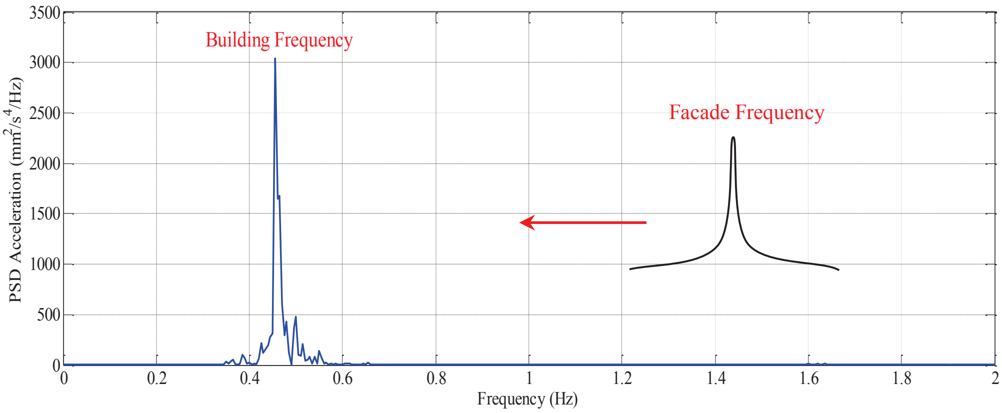


Figure 4.10 The first natural frequency of the mid-rise structure and façade system.

The results presented in this section were obtained from a 2D, 30 storey mid-rise structure with Conventional façade, compared with the Smart façade response under the same wind load excitation. In both conventional and smart façade cases, wind speeds from 12 m/s to 30 m/s with 50 years return periods are considered. The behaviour of the dampers is also plotted to present the difference due to nonlinear response caused by wind excitation. The Cumulative density function is used to demonstrate the different response in a better and more accurate way. It can be noted that the smart system could decrease the response of structures by up to 50% when subjected to 20m/s wind speed and similar reductions are also expected in lateral displacement of the structure (Figure 4.10). Figure 4.11, the curves are smoother and there are no spikes in displacement response.

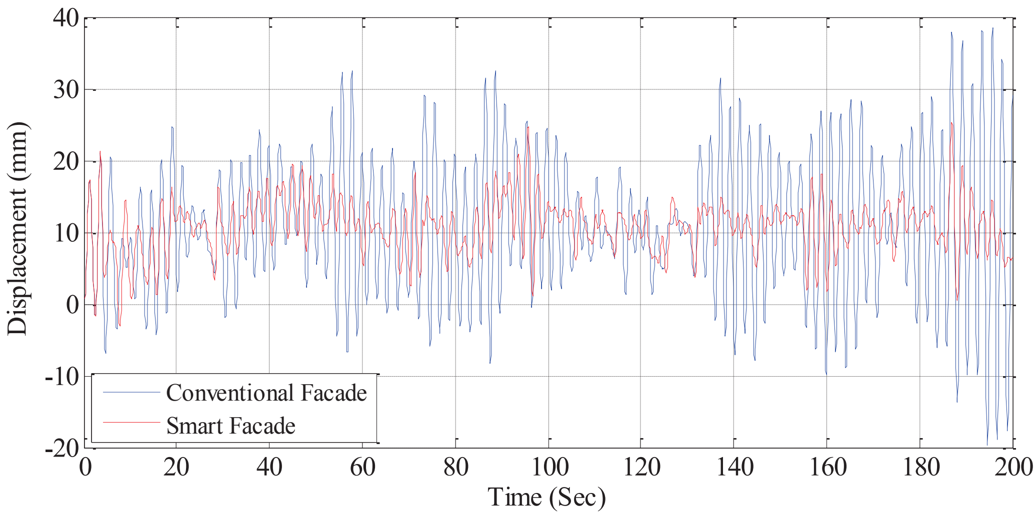


Figure 4.11 The displacement response of conventional façade vs smart façade (mean wind speed of 20 m/s)

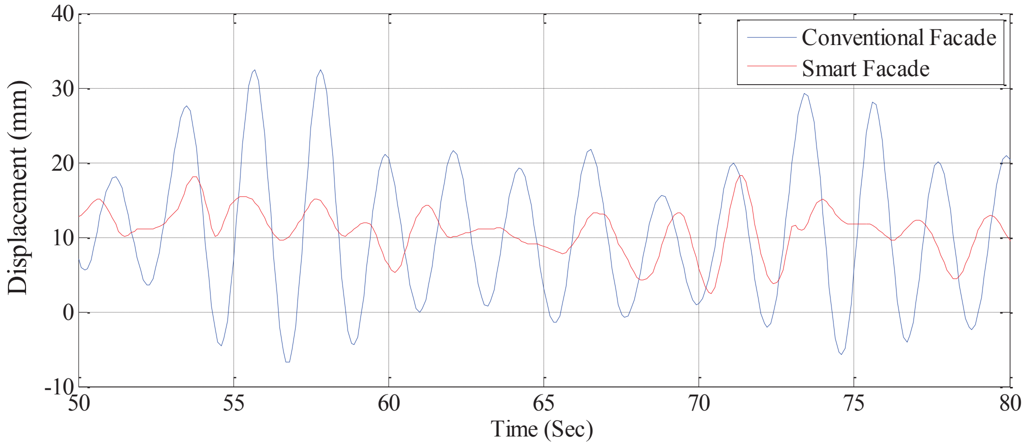


Figure 4.12 The displacement response of conventional façade vs smart façade from 50sec to 80sec (means wind speed of 20 m/s)

Figure 4.12 shows the behaviour of the damper subjected to 20m/s wind speed. As noted before, façade panel should vibrate at a certain frequency to have certain beneficial effect; in order to do so, having low stiffness could help the panel to reach the desired vibration frequency. Each time façade panel passes the soft slope it can reach the maximum dissipation phase through the dampers therefore; the number of times the panel crosses the soft stiffness part is another important criterion to be met.

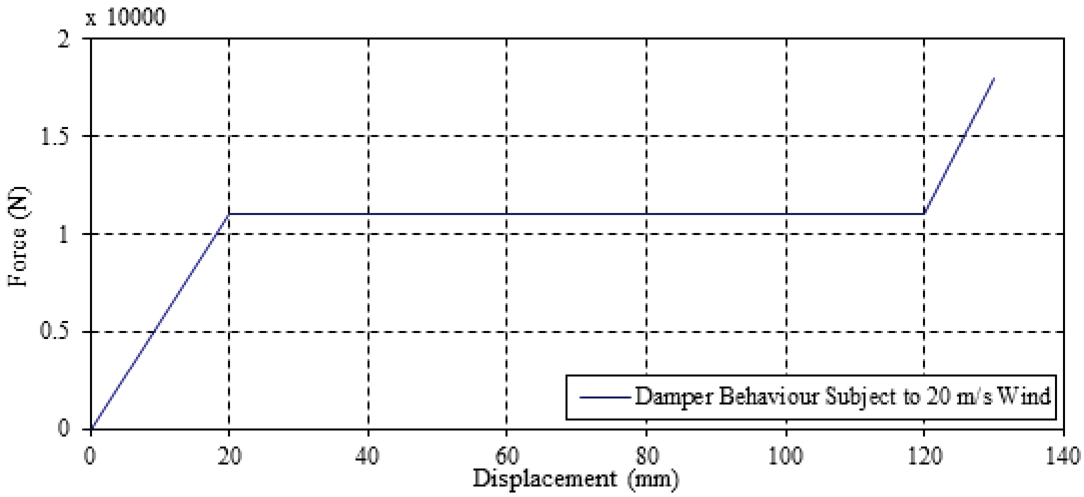


Figure 4.13 Behaviour of smart damper due to wind excitation (means wind speed of 20 m/s)

As stated earlier, spikes in the response curve are occurring due to sudden slope changes in the damper stiffness. By having a sharp end in the behaviour of damper the spike phenomena are inevitable. Although smoothing the sharp corners can lead to a smoother response, it does not eliminate the whole spikes phenomenon.

In probability theory and statistics, the cumulative distribution function (CDF), or just distribution function, evaluated at 'x', is the probability that a real-valued random variable *X* will take a value less than or equal to *x*. In other words, CDF(x) = Pr(X<=x), where Pr denotes probability. In this research to have better differentiation in the response of each case cumulative density function plots have been used.

For example, if 'x' is the acceleration response of the structure then F(x) is the chance that the response of the structure with smart damper façade will be smaller than the structure without smart damper facade. Therefore CDF(5)=0.8, which means that there's an 80% chance that a point selected at random will be smaller than 5 milli-g.

Cumulative Density Function is used in order to ensure that the system could decrease the response of the structure uniformly and also ensure that smart system can reduce the negative and positive picks in the same order, as shown in Figure 4.13.

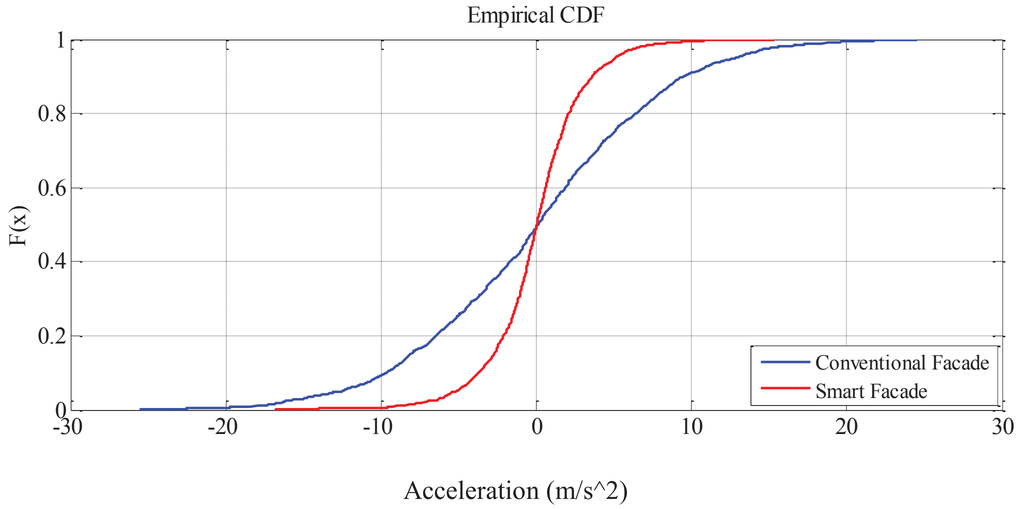


Figure 4.14 Cumulative density function of conventional vs smart façade response due to wind excitation (means wind speed of 20 m/s)

4.1.4 Sensitivity Analysis under yearly wind

Sensitivity analysis has been undertaken to assess the behaviour of the system under yearly wind. By observing the movement of the façade during a whole year and determining the number of days when this system works with full or partial efficiency could help a better and clearer understanding of this system. Based on this concept the following discussion shows the performance of the system in Sydney in 2012. Figure 4.14 shows the maximum daily wind speeds in 2012 in Sydney at the reference level of 10 m above ground. The maximum daily wind speed is measured during a whole year and divided based on different months.

For serviceability of tall buildings, particularly human comfort against wind induced accelerations, use of wind speeds with 1 year or 5 year return period is common. ISO code, Australian wind code also recommends these. Extensive work by Prof Bill Melbourne in 1980's and 90's also recommended the use of annual with speed and an upper limit of 10 mg acceleration for such winds. Hence, the yearly wind speed has been adopted in the current study.

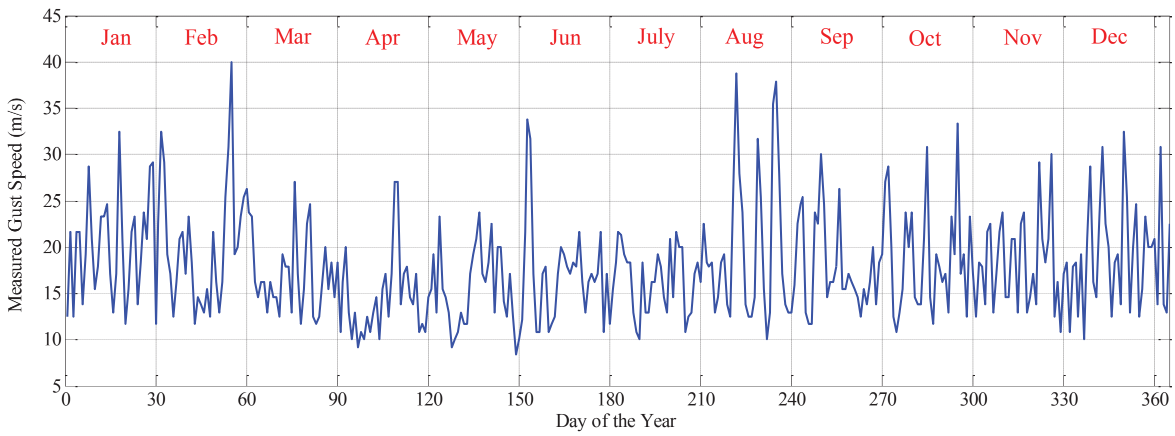
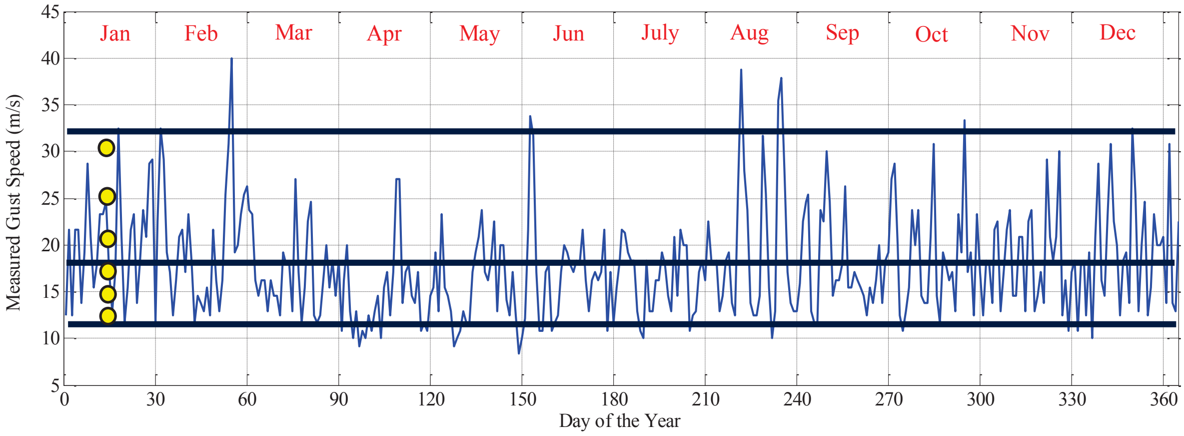


Figure 4.15 Max daily wind speeds in 2012 in Sydney (at 10m above ground)

4.1.4.1 Assessed Performance and Façade Displacement

The ultimate target of this research is to develop a system that can work in a wide range of yearly wind speeds. The system is meant to be activated and cover the wind speeds from 15-33m/s while being calibrated to 23m/s mean wind speed. Based on this target, an annual Sydney based wind is chosen to summarize the yearly behaviour of the damper system in Figure 4.15. The yellow circles in the graph show the mean wind speeds which are chosen to be analysed. The result of these analyses is presented based on top structural displacement and acceleration and also façade relative displacement.



Mean wind Speed	Structural Displacement	Structural Acceleration	Façade Displacement
Vm (m/s)	Sdisp (mm)	Sacc (mg)	Fdisp (mm)
12	12	5.2	±5
15	22	11.5	±9
18	26.1	12.3	±50
20	27	25	±55
23.3	45	30	±55
27	53	31	±60
30	67	32	±65
33	70	31	140 (Static)

Calibration wind speed from AUS/NZ with 1 year return period

Figure 4.16 Maximum daily wind speed in 2012 in Sydney (10m above ground)

The efficiency curve shows the performance of the system for different values of mean wind speed. The graph shows the efficiency of the device versus different mean wind speeds, in which the yellow circles represent the mean wind speed value of 12, 18, 20, 23.3, 27, 30 and 33 m/s . As seen in Figure 4.16, even a calibrated damper for 23.3 m/s mean wind speed, can never be perfectly calibrated. The real performance of the device is around 20 m/s which is less than what we expect based on our calibration. The performance range is one of the important criteria in every device. Although the performance of this device stays above 40% in a wide range of mean wind speeds [19 to 24m/s] this range needs to be extended and validated under more realistic loading patterns. Based on sensitivity analysis under wind speed of 16m/s we can say the system works as a conventional façade.

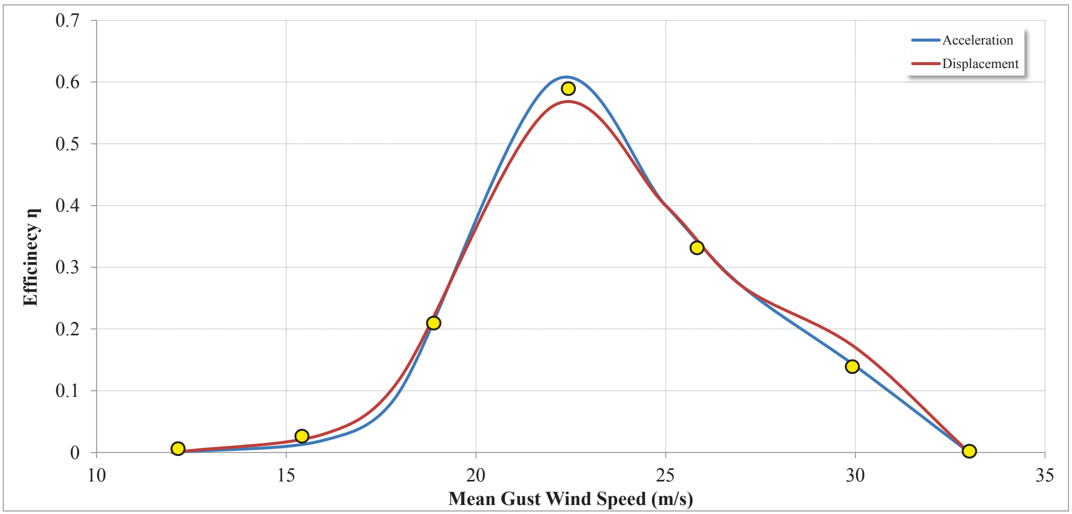


Figure 4.17 Performance spectrum of the system

Vibration of the facade could start from 17m/s which is based on our design value. In other words, this system works in a particular range and this range is adjusted based on the location and type of the structure. Considering mechanical limitations in façade panels and coupling with movable system, at least ±55mm is needed to be efficient. Beyond mean wind speeds of 30m/s we can also say it becomes a traditional façade (Figure 4.17).

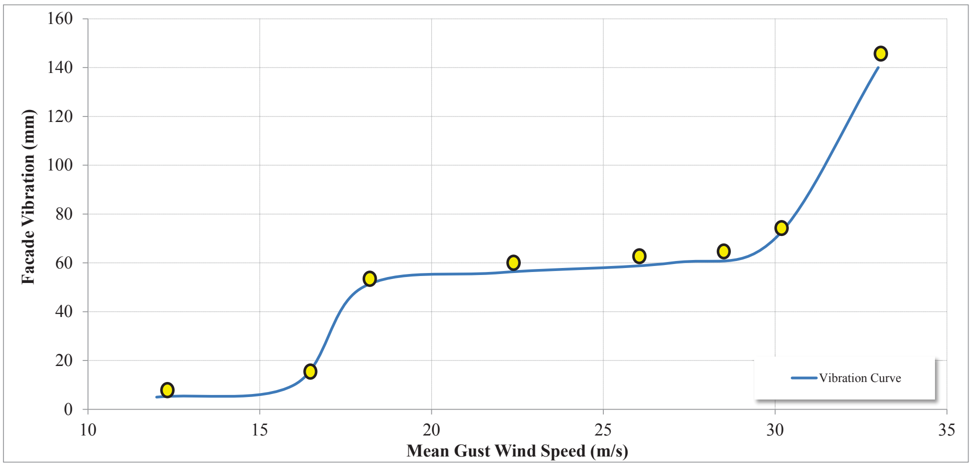


Figure 4.18 Façade vibration versus mean speed wind

Figures 4.18 and 4.19 present the sensitivity analysis performed to show the yearly efficiency of the device. The figures show that the system has an effect on acceleration and displacement of mid - rise structures during the year and when it moves it could be effective up to 60%.

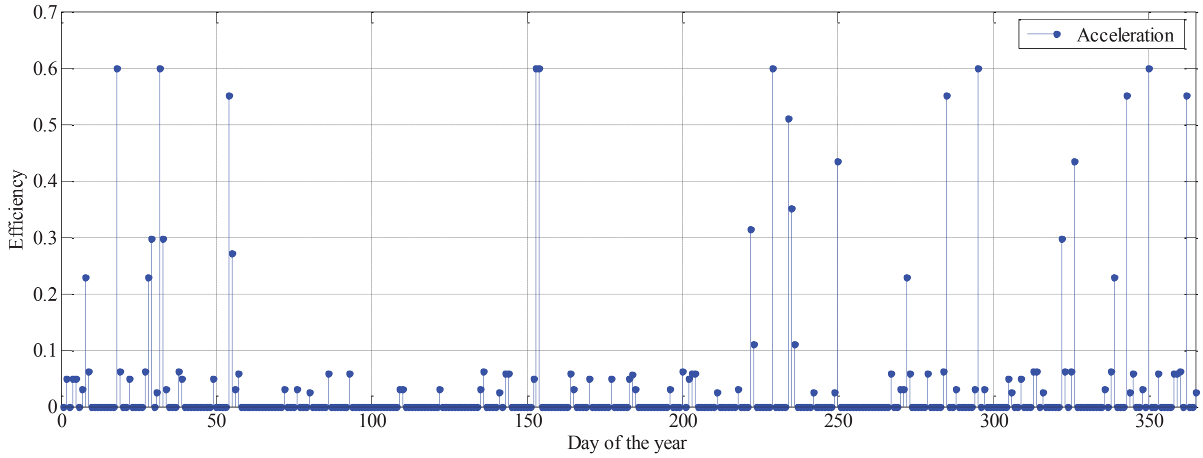


Figure 4.19 Efficiency of façade damper system during the year (Acceleration)

One of the most important aspects of the design procedure is how often does this device move and is it always efficient? Movement can always happen, but without any beneficial effect!

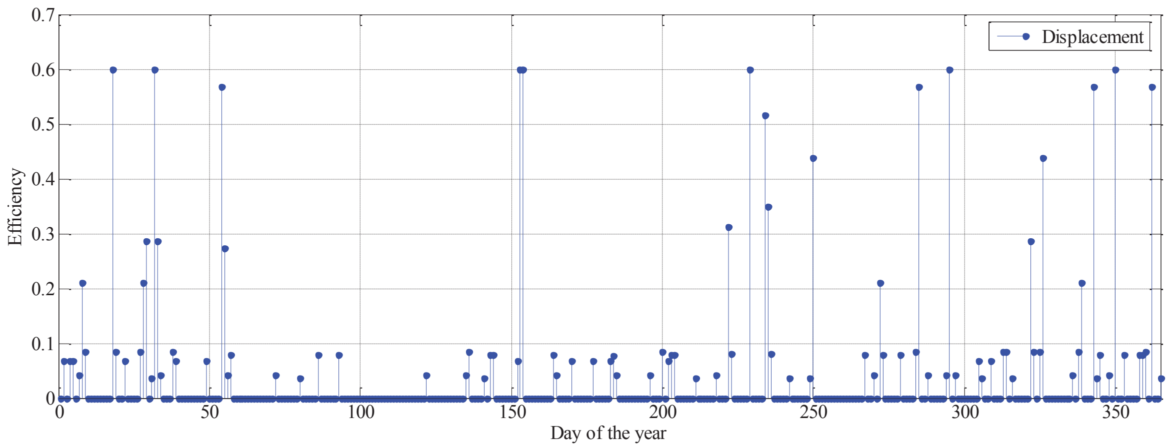


Figure 4.20 Efficiency of façade damper system during the year (Displacement)

A 3D graph is presented to show how efficient is this movement (Figure 4.20). There are some inefficient movements that can happen with less than 1% efficiency. As stated earlier, the system has a tremendous beneficial effect comparing to other available systems, although it needs better tuning to eliminate inefficient movements, and reducing such movements could improve the comfort level of the building occupants.

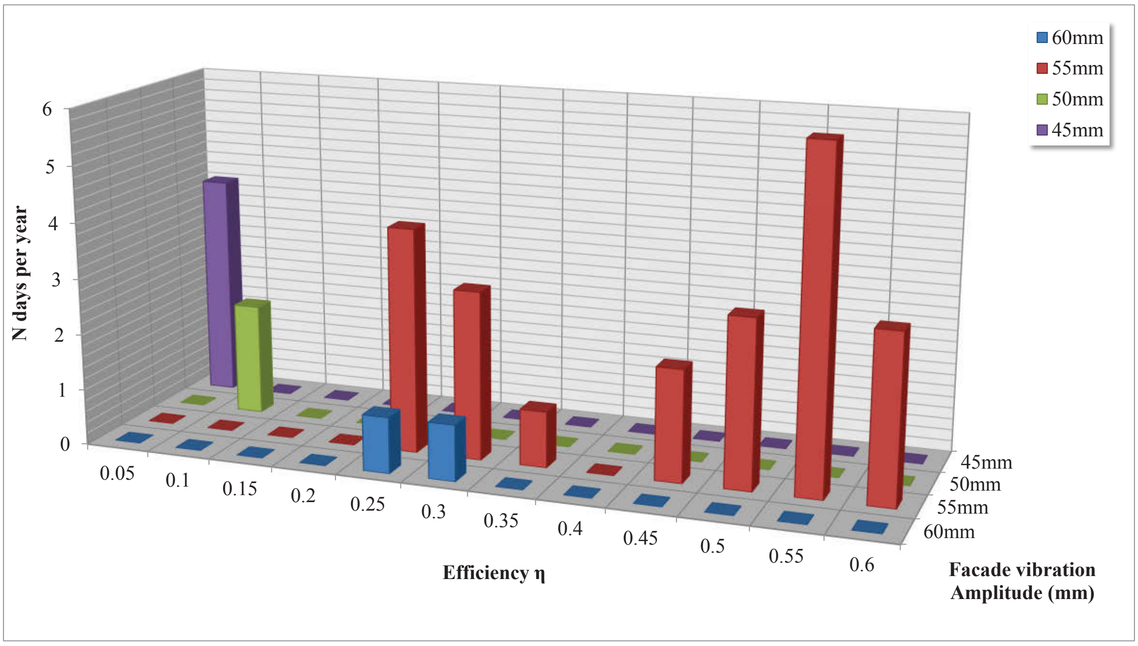


Figure 4.21 Efficiency of façade damper system during the year

4.2 Mid-Rise Structure 3D Analysis

The following model (Figure 4.21) represents the 3D mid-rise concrete structure which has been derived from Ansys software. In the finite element modelling, in order to carry the gravity load, line elements are used for columns and beams and in order for the structure to carry the lateral loads, X brace system is utilized.

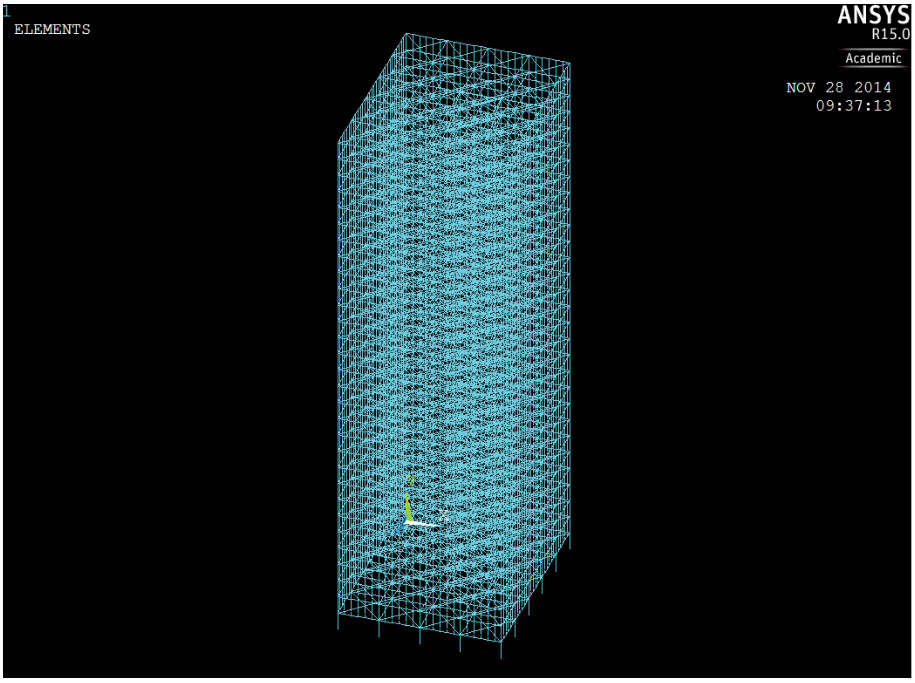


Figure 4.22 Finite element model of 3D Structure model

The damper façade system is designed in a way that only the top one third of the façade panels move. By this movement, the strains and the stresses of the panels and the damper from which the panels hang, increase. This is illustrated in the following figure. According to figure 4.22 when the wind blows onto the structure, the afore-mentioned one third facades react and start to move. This vibration is effective when the wind speed is 23m/s which is actually the design mean wind speed. The movement of the façade panels has been magnified for better clarity. In actual cases the range of this movement is between 20mm to 150mm.

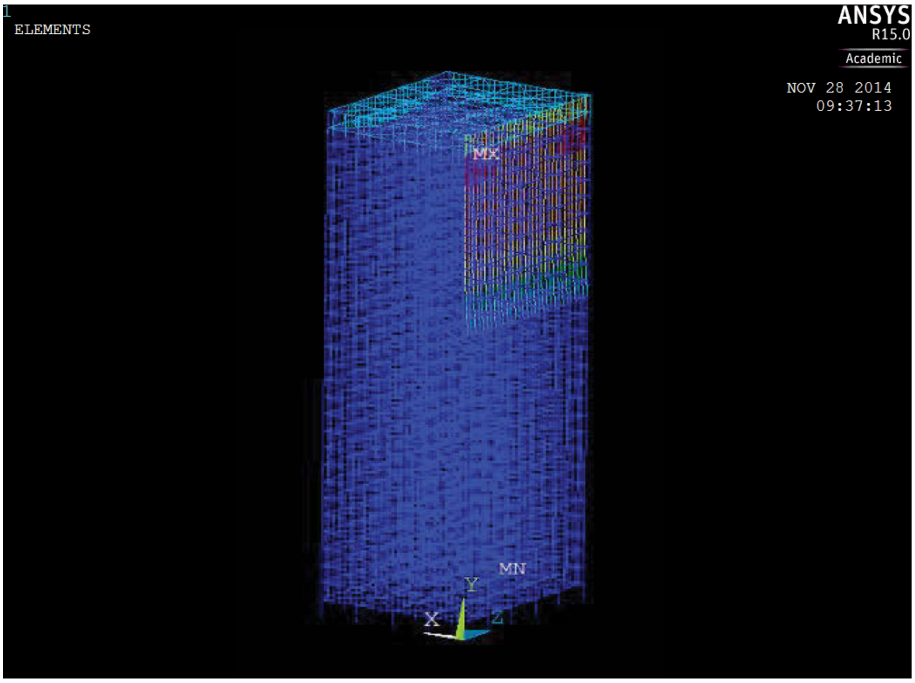


Figure 4.23 Finite element model of 3D Structure model subjected to the wind load

Due to time consuming analysis of 3D structures, sensitivity analysis is based on four different mean wind speeds starting from 15m/s (as a low wind speed) to 30m/s (as a high wind speed).

Upon increasing the mean wind speed from 15m/s to 20m/s the system enters phase two behaviour causing the panels to vibrate in a certain manner (Figure 4.23). This vibration and the change from phase one to phase two activates the damper system

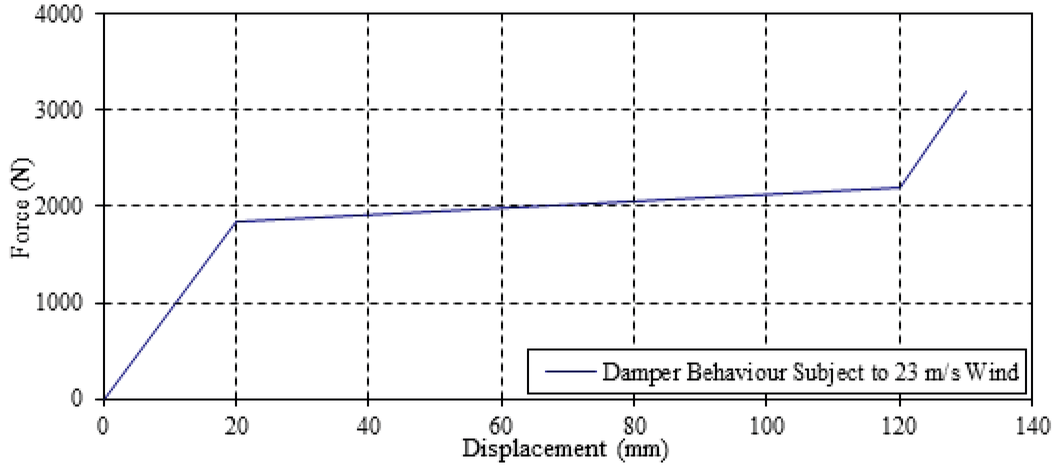


Figure 4.24 Behaviour of smart damper due to wind excitation (Means Speed 23 m/s)

As indicated in Figure 4.24, the optimum performance of the damper façade system happens when the mean wind speed reaches 23m/s. In fact, at this particular wind speed the system is fully activated. This means that the system has gone through the three phases changing its stiffness.

These changes help the system to reach its optimum performance at 23m/s mean wind speed. Taking this optimum performance into account, the smart system decreases the response of the structure by 50% (Figure 4.24).

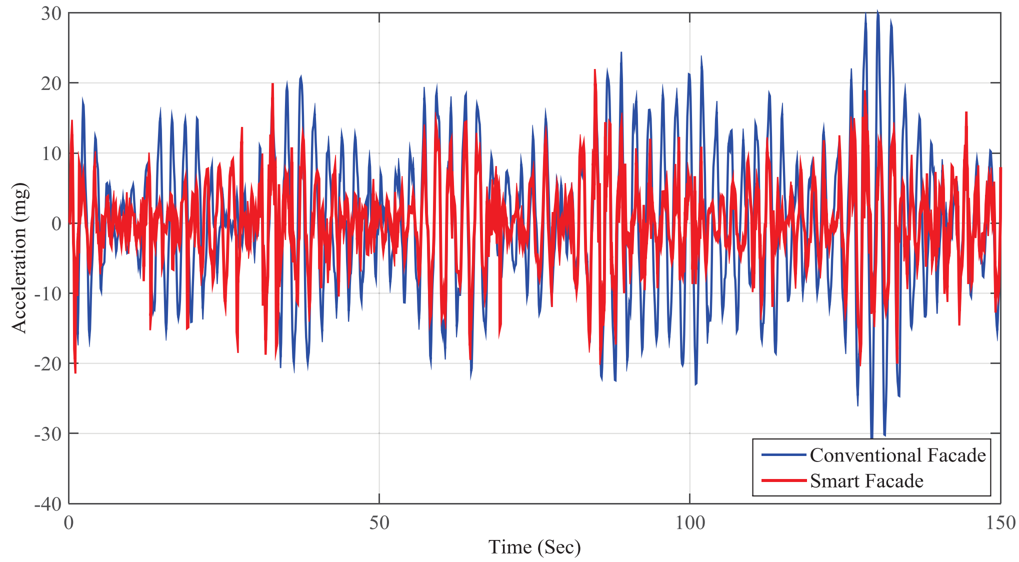


Figure 4.25 Acceleration response of structures with and without damper façade system subjected to 23m/s mean wind speed

In summary, the sensitivity of the smart façade damper system subjected to variable mean wind speeds has been investigated in the 2D and the 3D structure. In either case the smart system reduces the response of the main structure and proves effective. The 3D model which in fact corresponds to real situations indicates that the system is sensitive to any slight changes in mean wind speed.

4.3 High-Rise Structure 2D Analysis

The race towards building skyscrapers has been met with numerous challenges. Unfortunately, super tall buildings are often accompanied by increased structural flexibility and a lack of adequate inherent damping. This leads to an increase in the structure's susceptibility to wind action. For taller buildings the impacts due to lateral forces become much greater.

Various state-of-the-art technologies are now available that can be used in the design and construction of new buildings in order to improve their dynamic behaviour. The acceptance of innovative systems in building structures depends on a combination of performance enhancements versus construction costs and long-term effects. New innovative devices need to be integrated into these structures, with a realistic evaluation of their performance and impact on the structural system. Moreover, their ability to operate in the long term is an important factor that must be considered.

It should be noted that environmental loads are likely to have a huge bearing on the vulnerability of the exterior face of façade systems and load transmission to the primary structure. Moreover, due to the significant usage of glass in buildings in recent decades, an increasing emphasis has been placed on protecting them during typhoon events. This emphasis and focus on the safety of façade systems has led to their high cost of installation and, repair and in their reduction of potential for posing severe safety risks to people during typhoon activities. (Connor 2003). The loss of valuable and prime space coupled with an initial cost of installing large sized damper systems have been accepted unwillingly by building owners and any viable alternative system to dissipate wind forces will be welcomed by them. The emphasis of this research is, therefore, placed on evaluating the interrelation between new technologies in the field of structural façade dynamics.

4.3.1 Structural Modelling

In order to evaluate the wind response of the proposed system, the main structure and the facade system are simplified and modelled as line elements using finite element method for analysis. The system consists of primary structural components (Beams, columns, and shear wall) representing the main building structure, and the façade panels simplified as a vertical line.The Figure 4.26 show the floor plan and the major input data for the definition of the mid-rise test case buildings.

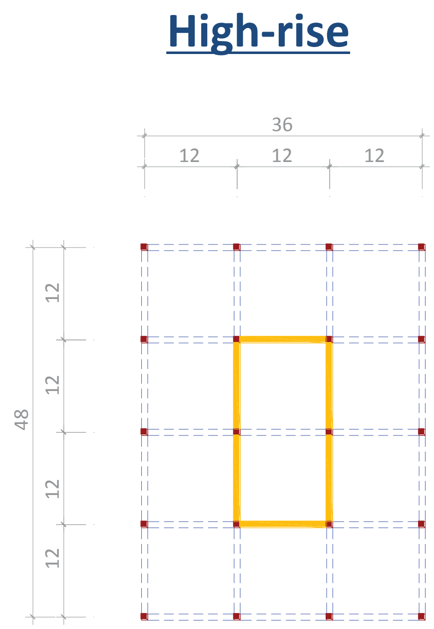


Figure 4.26 High-rise and high-rise test case building floor plant

Figure 4.27 shows the finite element model of a 76 storey (300m height) structure in ANSYS. The structure is designed based on Australian and New Zealand standard to obtain the shear wall section.

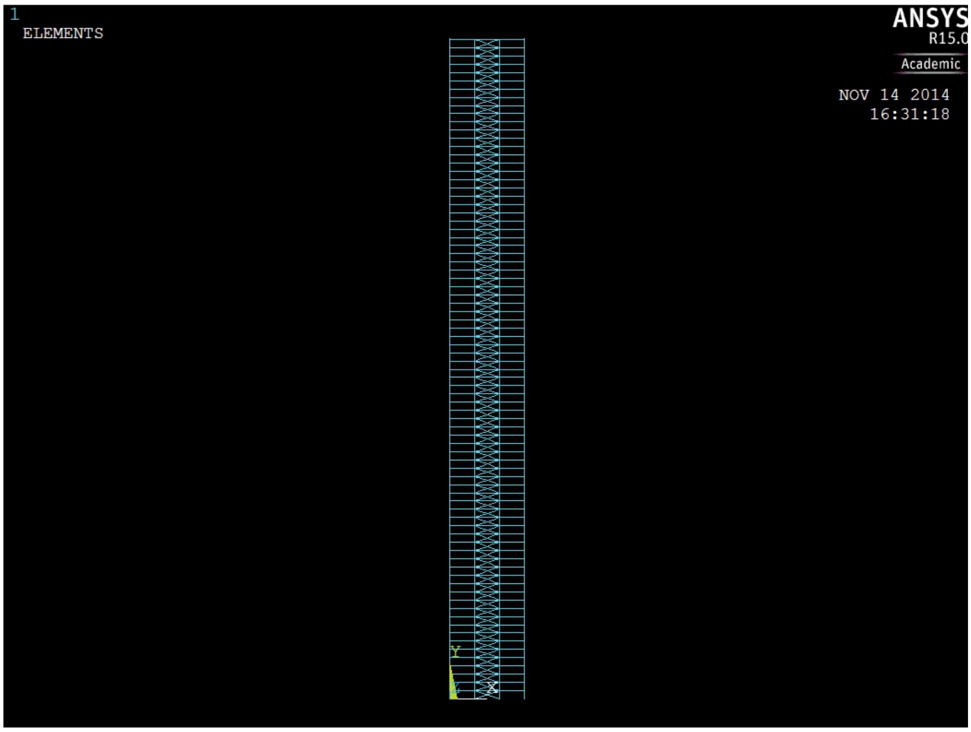


Figure 4.27 Finite element model of Structure with shear wall using brace system

In order to ensure that the structure subjected to lateral force is in line with the Australian New Zealand code, 5% of the total weight of the primary structure is applied as a distributed lateral force (Figure 4.26). The acceptable lateral deformation of tall buildings is limited to H/500 (H is the building height) which is greater than the maximum deformation shown in Figure 4.26.

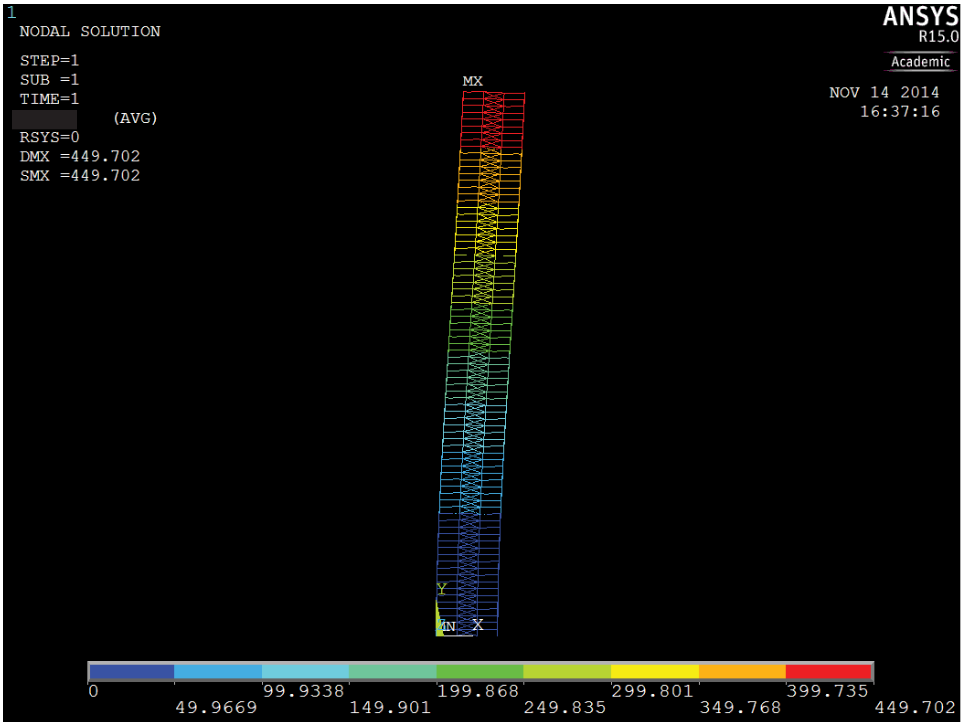


Figure 4.28 Finite element model of Structure subjected to lateral static load

Connection properties concerning stiffness and damping have been modelled and varied to achieve the appropriate response. For achieving optimal performance of the proposed system, the façade connection frequency is tuned to the primary mass frequency. Dynamic force applied to the secondary mass and through the connections, between the primary and the secondary masses, is transferred to the main frames. The outer skin mass is assumed to be around 1% of the primary structure mass. As Figure 4.27 illustrates, the distance between the facade panel and the primary structure is considered to be 200 mm to provide enough space for damper installation but this can be changed based on the bracket design. As shown, the façade panels are attached to the primary building by some brackets. These brackets are assumed to be flexible in order to absorb maximum external energy induced by the wind force and are considered as a non-linear damper element in ANSYS modelling.

With due regard to our optimisation we consider movable façade in the top one third of the structural height. The whole system hangs off each level with brackets which could move in the axial direction, but they are fixed in the vertical direction. The joint between façade panels on each floor is assumed as a pinned joint and they are attached to each other from above and below by these pinned joints to move freely during the application of environmental forces. The joint is located 900 mm above the floor slab level on each floor. The details of façade connection, attached to the main structure, are illustrated in Figure

4.27As shown, each panel is attached by a bracket to floor slab. The stack joint is designed to resist lateral loads while the two floor anchors resist gravity and lateral loads. One of the two floor anchors allows movement in a plane with the unitized system.

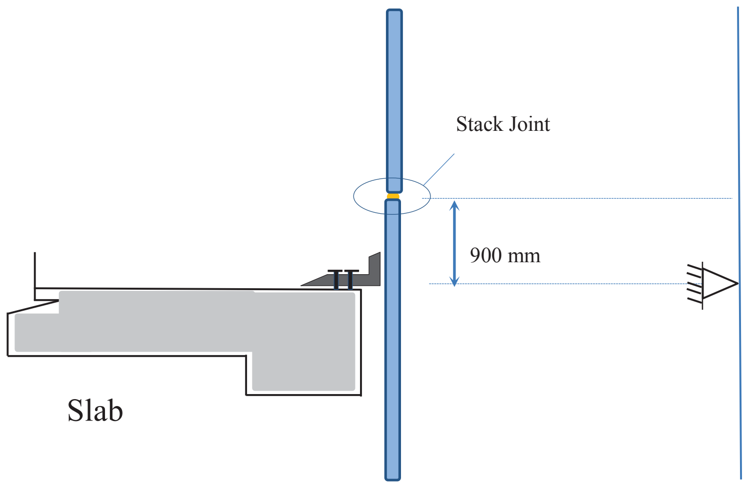


Figure 4.29Detail of façade connection to the primary structure and modelling assumption in ANSYS

In Table 4.5, the modulus of elasticity, Poisson's ratio and density of materials used for façade components in ANSYS modelling are shown.

Table4.5 Properties of façade system components

Façade Sections	Modulus of Elasticity (kN/mm^2)	Poisson's Ratio	Density (kN/m^3)
Glass	62.76	0.2	22
Aluminium	68.64	0.33	27

Concrete floor material properties used in ANSYS modelling are presented in Table 4.6 for the high-rise structure. These values were obtained from common material properties used by Australian construction companies.

4.26).

Table4.6 Material properties of main high-rise structure

Compressive Strength (MPa)		Modulus of Elasticity (GPa)	Poisson's Ratio	Density (kN/m^3)
Column	80	30	0.2	24.5
Beam	60			
Slab	50			

The primary high-rise structure damping ratio is assumed to be 0.5% of critical damping, which is within the range of statistically reasonable values based on measured natural damping ratios for high-rise buildings. As shown in Table 4.7, the bare frame has a frequency equal to the façade system with movable façade.

Table4.7 Structural model dynamic properties

Structure Type	Period (Sec)	Frequency (Hz)
Bare Frame	6.25	0.16
Frame with Movable facade	6.25	0.16

In order to use façade as a damper system which could be effective in terms of response of the main structure, one needs a concept. Sensitivity analyses are performed to find a way to reduce the responses. Two different concepts are considered to reduce the response of high-rise structures as follows.

4.3.2 Concept for High-Rise Buildings

Although high-rise buildings subjected to wind loads are also dominated by first natural frequency when it comes to changing the response of the structure the second natural frequency mode may also be large enough to be tuned to the smart damper system frequency in order to be effective. In high-rise buildings the second natural frequency of the building has a contribution of 30 to 60% to the overall Power spectral Density (PSD) acceleration (Figure 4.28).

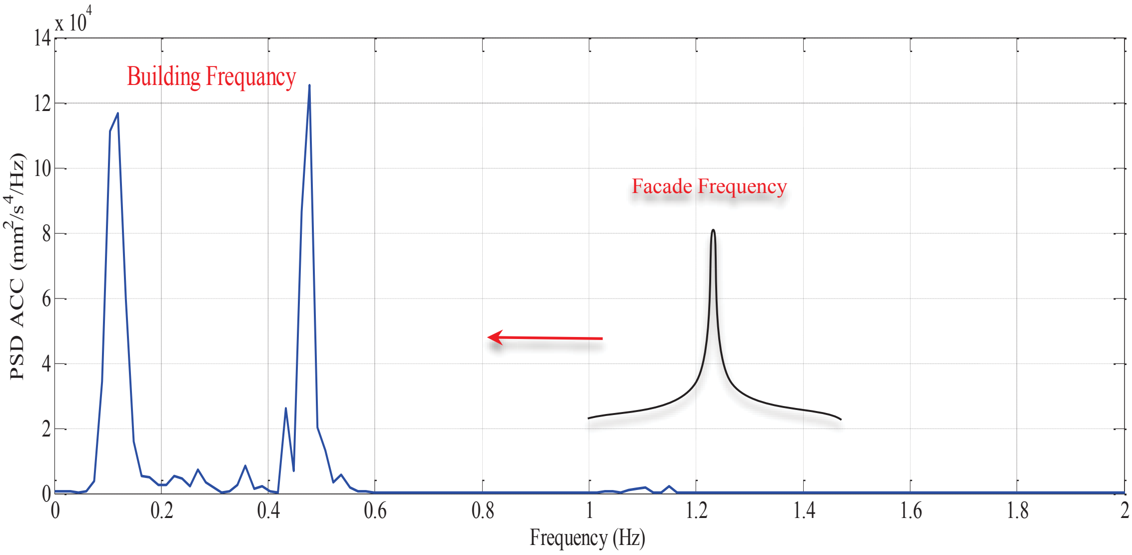


Figure 4.30 The first two natural frequencies of the high-rise structure and façade system

The sensitivity analysis shows that focusing on higher structural modes and other frequencies is not useful and using them as TMD cannot lead to any beneficial effects in terms of acceleration or displacement control of the main structure. Therefore, in order to have a significant TMD effect, the façade must vibrate in the first or the second natural frequency of the building with enough energy (mass and displacement).

An effective TMD can be achieved by tuning the device to the second frequency, paying attention to being far enough from the first frequency in order to avoid detrimental effects.

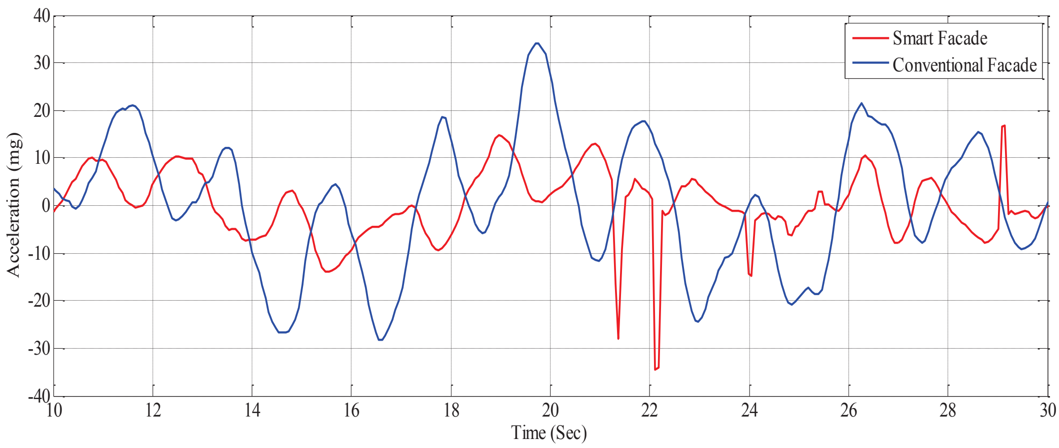


Figure 4.31 High-rise structure with traditional façade system response tuned to the second mode versus smart façade system

Due to the sudden changes in the stiffness of damper system and the huge difference in stiffness ratio, some spikes in the response are observed. It can be noticed that in the response of high-rise building systems these spikes are more common and the amplitude could be relatively twice than the actual accelerations. It can be concluded that the damper façade system could be effective if the panel frequency is tuned to vibrate close to the second dominant mode of high rise building (Figure 4.29).

A sensitivity analysis with regard to different mean wind speed values has been conducted. The following figures show the response of the system and the efficiency ratio when subjected to wind loads.

Increasing the mean wind speed to 23 m/s, forces the damper to move to the physical limitation of design which leads panels to vibrate in the desired frequency (Figure 4.28). As stated earlier, if the vibration frequency of the façade system is close to the second natural frequency of the structure, the response can reduce by up to 50% of the response of the main structure. As shown in Figure 4.30, the smart damper subjected to the 23 m/s mean wind speed could reach the required frequency which allows the smart system to reduce the response of the structure by up to half. The comparison of the acceleration response of smart and conventional façade systems is shown in Figure 4.31.

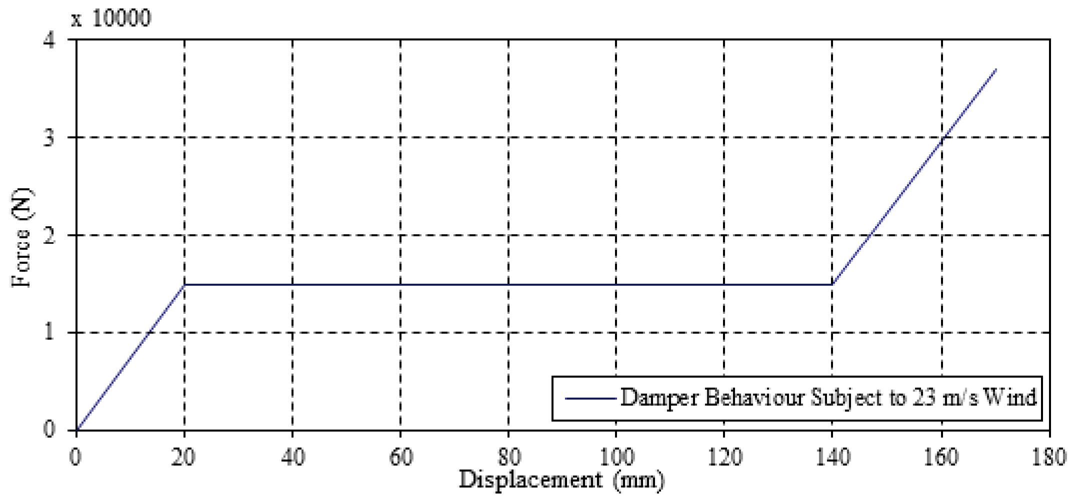


Figure 4.32 Behaviour of smart damper due to wind excitation (mean wind speed of 23 m/s)

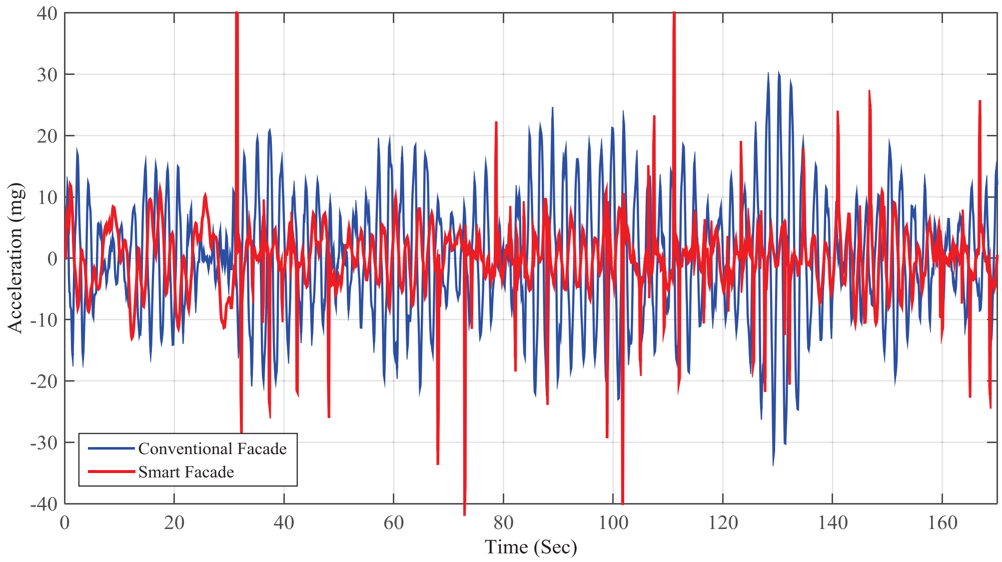


Figure 4.33 The acceleration response of conventional façade vs smart façade (mean wind speed of 23 m/s)

Distribution response functions shown in Figure 4.32, demonstrate the beneficial effects trend of the system with and without the smart facade. The blue curve represents the structure equipped with the conventional façade and the red curve represents the behaviour of the main structure equipped with a smart damper system subjected to 23m/s mean wind speed. It is worth noting that, damper façade can control the pick acceleration to be under 10 mg which is the human comfort threshold, although spikes phenomena occurred when the mean wind speed increased due to the sudden change of stiffness ratio.

It is worth noting that spikes on the results shown in Figure 4.33 is purely a numerical issue and does not occur in reality. Hence the spikes could have been smoothed out after the analyses by removing the spikes but the numerical results were left “as is” to show the actual results obtained from the analyses. The spikes are most likely due to nonlinear behavior of the dampers. The force displacement curve of damper behavior is modelled up to 150mm displacement including some non-linearity but in the nonlinear solution ANSYS will do linear approximation to generate the curve for further force, therefore, some numerical errors can appear in some of the results. It is worth mentioning that there are no spikes in the response of Midrise structure.

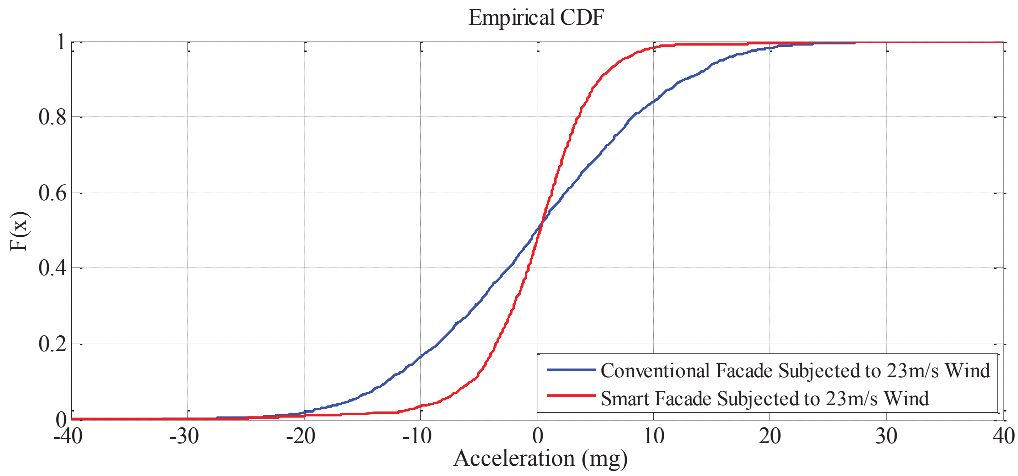


Figure 4.34 Cumulative density function of conventional vs smart façade response due to wind excitation (mean wind speed of 23 m/s)

4.4 High-rise Structure 3D Analysis

The results presented in this section were obtained from the 3D high-rise structure with conventional façade and is compared with the smart façade response under the same wind load excitation. For both conventional and smart façade, wind speed was started from 12 m/s and increased to 30 m/s. The behaviour of the dampers is also plotted to present the difference in nonlinear response due to wind excitation.

In this research a 3D model has been used in order to conduct both along wind and cross wind analysis of the structure. However, it was later decided to limit the study to along wind motion due to the fact that a reliable cross wind spectrum to simulate equivalent cross wind forces did not exist and these could obviously be done via wind tunnel studies which was beyond the scope of the work in term of the required time and cost. Therefore the 3D along wind results were used as a check for 2D along wind forces.

As stated earlier smart dampers are designed based on daily wind speed which, in Sydney area the mean speed is close to 20 m/s. It is expected that this range of wind speed will lead to dampers to be activated and make the panels move in a certain frequency. As shown in Figure 4.33, due to the panel movement in a certain frequency, smart system can reduce the top acceleration by about 40%. It could be demonstrated that smart system has also beneficial effects in terms of top displacement (Figure 4.34).

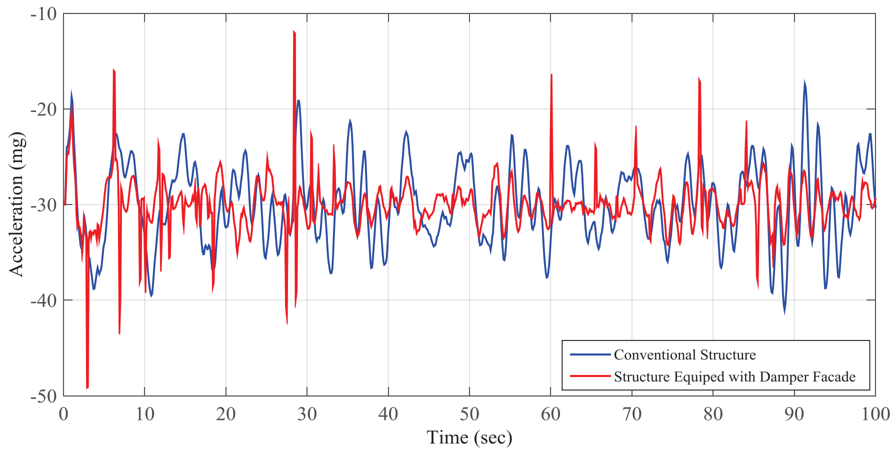


Figure 4.35 Acceleration response of conventional façade versus smart façade (Means Speed 20 m/s)

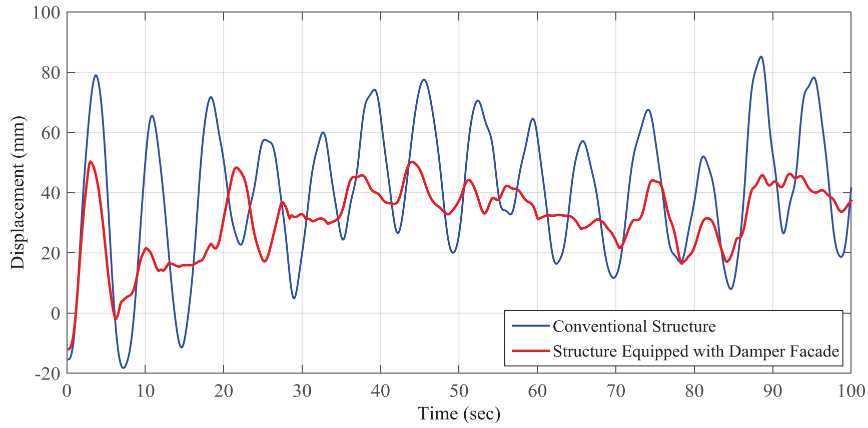


Figure 4.36 Displacement response of conventional façade versus smart façade (Means Speed 20 m/s)

Using right stiffness and having sufficient number of crossings lead to better performance of the smart system. Nevertheless, the spikes in the response of the structure shown in Figure 4.35 lead to having larger responses in the two ends of the density function spectrum.

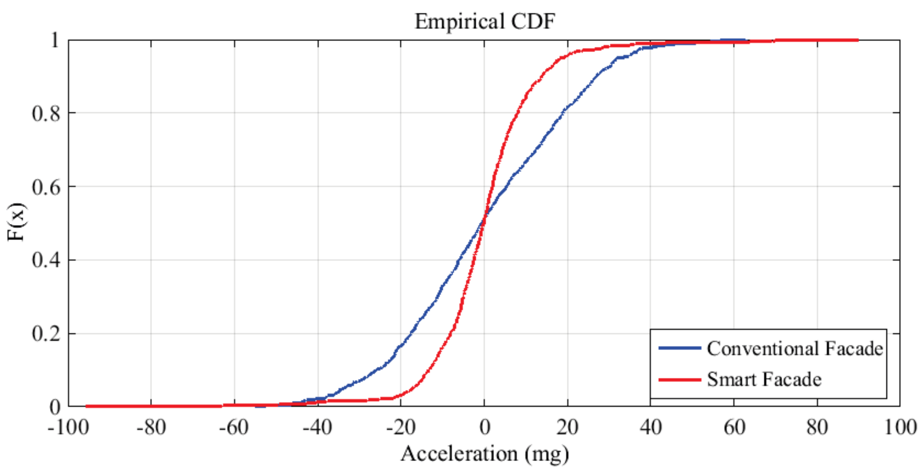


Figure 4.37 Cumulative density function of conventional vs smart façade system due to wind excitation (mean wind speed of 20 m/s)

Figure 4.36 shows the behaviour of the damper for 20 m/s wind speed. As noted before, façade panels should vibrate at a certain frequency to have certain beneficial effects and in order to do so having low stiffness could allow this vibration, although the number of crossing on the soft part is the other important criterion which has to meet the needs.

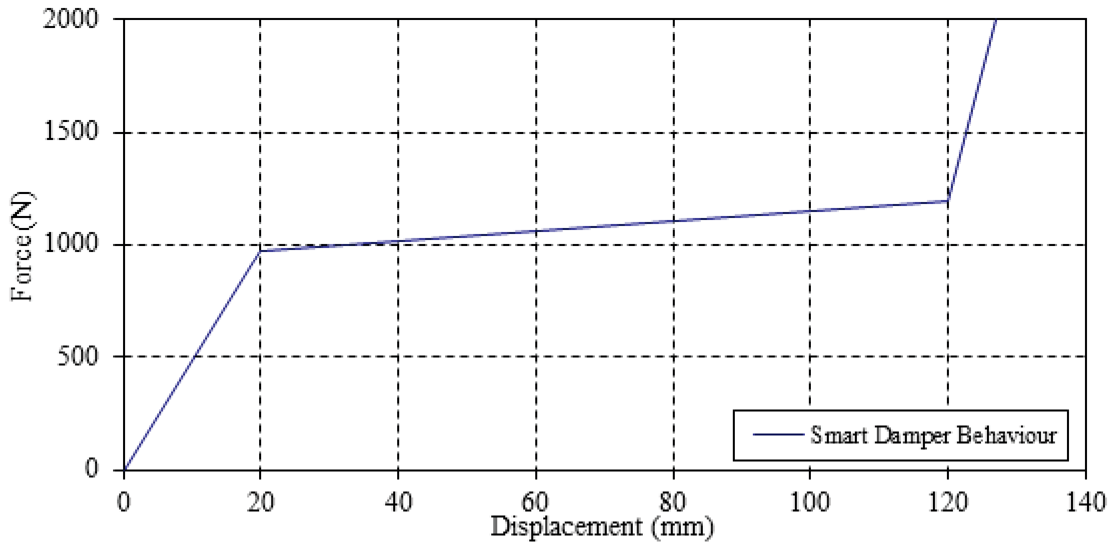


Figure 4.38 Behaviour of smart damper due to wind excitation (mean wind speed of 20 m/s)

4.4.1 Conclusions of the Preliminary Analyses

Façade acts as a protector for every building from controlled interior and harsh exterior as well as building identifiers as a result of their design. Regarding the façade system, this chapter investigated another potential functional dimension of double skin façades in buildings as lateral motion control devices. Having these systems could decrease top accelerations by up to 50% and enhance the level of serviceability of mid-rise building which is the most important aspect in the design of these types of buildings under wind loads.

The new Damper Façade System (DFS) design utilises the non-linear hysteresis phenomenon associated with viscous dampers for the façade connections between inner and outer skins. The system is controlled by a sensor network which monitors the behaviour of the outer skin façade during extreme wind conditions. The study has shown that the proposed DSF system can reduce the wind-induced vibrations by up to 30%. As a result, significant material savings of structural systems can be achieved as well as significant gain in leasable area compared to conventional systems

Chapter 5

5 Behaviour of Double skin façade in suppressing wind loads

5.1 Introduction

In recent decades, buildings with significant usage of glass are becoming more common. The development of non-load bearing curtain walling technology around the turn of the 20th century, along with double skin façade (DSF) system, which have substantial cavity space between the inner and outer façade layers, have increased interest in these systems with the aim of fully exploiting their potential. Building façades generally perform as environmental medium between the controlled interior and harsh exterior as well as building identifiers through their aesthetic design.

On the other hand an increasing emphasis has been placed on controlling structural dynamic response of wind sensitive buildings during moderate to severe winds.

The use of space frame and mega-frame concepts, outrigger trusses, belt trusses and band-aid type stiffening systems can offer additional resistance to wind loads (Kareem 1992). Other alternatives include modification of the structural mode shapes to increase the mass participating in the dynamics of building in the fundamental mode.

Kareem (1992) proposed the concept of isolation in the mountings of the cladding to the structural system. Buildings are isolated from earthquake excitation by employing isolator bearings between the building and the foundation and a similar concept is proposed for cladding. The integrated effects of the unsteady aerodynamic loads acting on the cladding are transferred to the frame which results in building motion. If the cladding is connected to the frame by an isolation mounting, then the aerodynamic loads transferred to the frame will be reduced and consequently the building motion will be reduced as well. In order for this mounting to be effective, the ratio of excitation frequency to the natural frequency of the cladding should be greater than square root of two (Kareem 1992). In this situation the mounting system is more effective without any damping.

The proposed system can be materialized by dividing the cladding on the building envelope into several segments. The preliminary calculations of (Kareem (1992) suggest that such a mounting system will be quite soft and pneumatic mounts may be an appropriate choice here. Such an installation may cause the cost of a cladding system to be, however, very high. This can be overcome by using these systems in staggered configurations and the remaining portions of the building envelope may utilize conventional cladding. The staggered arrangement has been proposed to help reduce the correlation of wind-induced pressure which in turn would result in lessening the integrated loads.

Moon (2005) shows that dynamic motion of tall buildings can be reduced, for example, by more than 50% when the DSF façade connectors are designed to have about half of the primary structure frequency. However, there exists a design challenge which is the excessive and extreme motion of the DSF outer skins, which would disturb occupants through visible cues, and would potentially undermine the ventilation system intended by DSF systems through pumping cavity air around the building.

5.2 System Modelling

A simplified model is used in order to demonstrate the behaviour of the proposed system. The complex primary structure with an outer skin facade could be modelled as a two degree of freedom system as shown in Figure 5.1, where primary mass represents the structure (including the inner skin mass) and the secondary mass represents the outer skin. Usually this kind of modelling is used to present a tuned mass damper (TMD) system, although there is a different mechanism to apply the load in these cases.

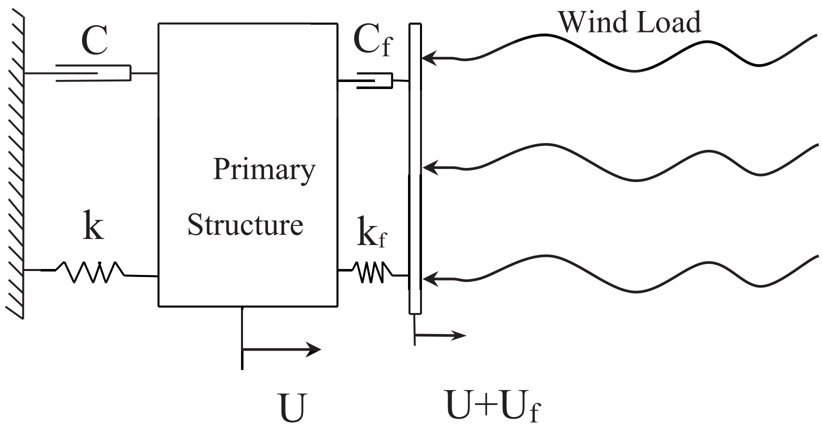


Figure 5.1 Simplified model of the primary structure and façade system connected by movable brackets

Loads on the tuned mass damper system, are applied to the primary mass and then transferred to the secondary mass. The connection between the primary mass and the secondary mass should be chosen so that the TMD mass frequency is similar to structural frequency (Den Hartog (1956)), however, in the proposed system here, the loads are applied to the secondary mass and then, through the proposed connection will be transferred to the primary mass. This difference in load transfer makes tuned mass damper formulations inapplicable.

Connection properties concerning stiffness and damping have been modelled and varied to achieve the appropriate response. For achieving the optimal performance of the proposed system, the connection frequency is tuned to the primary mass frequency. Dynamic forces are applied to the secondary mass and through the connections, between the primary mass and the secondary mass, are transferred to main frames. The outer skin mass is assumed to be around 1% of the primary structure mass.

5.3 Dynamic Responses of the System

Below are the governing equations of the system shown in Figure. 5.1:

$$m\ddot{u} + c\dot{u} + ku = \mathrm{c}_f\dot{u}_f + k_f u_f \tag{5.1}$$

$$m_f(\ddot{u}_f + \ddot{u}) + c_f\dot{u}_f + k_f u_f = p \tag{5.2}$$

where $m =$primary structure mass; m_f =DSF outer skin mass; k =primary structure stiffness; k_f =DSF connector stiffness; c =primary structure viscous damping parameter; c_f =DSF connector viscous damping parameter; p =applied dynamic loading; u =primary structure maximum lateral displacement; and u_d =DSF outer skin maximum lateral displacement. It is convenient to work with the solution expressed in terms of complex quantities. The force is expressed as:

$$p = \hat{p}e^{i\Omega t} \tag{5.3}$$

Where $\Omega =$forcing frequency and $\hat{p}$ is a real quantity representing the loading amplitude. The response is taken as

$$u = \bar{u}e^{i\Omega t} \tag{5.4}$$

$$u_f = \bar{u}_f e^{i\Omega t} \tag{5.5}$$

Where $\omega =$natural frequency of the primary structure, and where the response amplitudes, $\bar{u}$ and $\bar{u}_f$, are considered to be complex quantities. Then the corresponding solution is given by either the real or imaginary parts of u and u_f. Substituting Eqs. (5.3)–(5.5) into the set of governing Eqs. (5.1) and (5.2) results in

$$-\Omega^2 m\bar{u} + i\Omega c\bar{u} + k\bar{u} = i\Omega c_f \bar{u}_f + k_f \bar{u}_f \tag{5.6}$$

$$-\Omega^2 m_f(\bar{u}_f + \bar{u}) + i\Omega c_f \bar{u}_f + k_f \bar{u}_f = \hat{p} \tag{5.7}$$

Considering the following notations:

$$\omega^2 = \frac{k}{m} \tag{5.8}$$

$$c=2\xi\omega m \tag{5.9}$$

where ξ =primary structural damping ratio, and

$$\omega_f{}^2 = \frac{k_f}{m_f} \tag{5.10}$$

and ω_f =natural frequency of the DSF outer skin, k_f =Stiffness of the brackets which is a variable and a function of the input frequency.

$$c_f = 2\xi_f \omega_f m_f \tag{5.11}$$

where ξ_f =façade connector damping ratio. Defining $\bar{m}$ as the DSF outer skin mass to primary mass ratio,

$$\bar{m} = \frac{m_f}{m} \tag{5.12}$$

and defining f as the DSF outer skin frequency to primary structure frequency ratio.

$$f = \frac{\omega_f}{\omega} \tag{5.13}$$

and defining ρ as the forcing frequency to primary structure frequency ratio,

$$\rho = \frac{\Omega}{\omega} \tag{5.14}$$

Then the corresponding Frequency Response Functions (FRF) can be obtained by derivation from the equations of motion as follows.

$$H = \frac{\sqrt{f^4 + 4f^2\zeta^2{}_f\rho^2}}{\sqrt{(f^2\overline{m}\rho^2 - \rho^4 + \rho^2 + f^2\rho^2 - f^2 + 4\zeta\rho^2\zeta_f f)^2 + (2\rho^3\zeta_f f + 2\zeta\rho^3 - 2\zeta\rho f^2 - 2f\zeta_f\rho + 2\overline{m}\rho^3\zeta_f f)^2}} \tag{5.15}$$

$$H_f = \frac{\sqrt{(\rho^2 - 1)^2 + 4\zeta^2\rho^2}}{m\sqrt{(f^2\overline{m}\rho^2 - \rho^4 + \rho^2 + f^2\rho^2 - f^2 + 4\zeta\rho^2\zeta_f f)^2 + (2\rho^3\zeta_f f + 2\zeta\rho^3 - 2\zeta\rho f^2 - 2f\zeta_f\rho + 2\overline{m}\rho^3\zeta_f f)^2}}$$

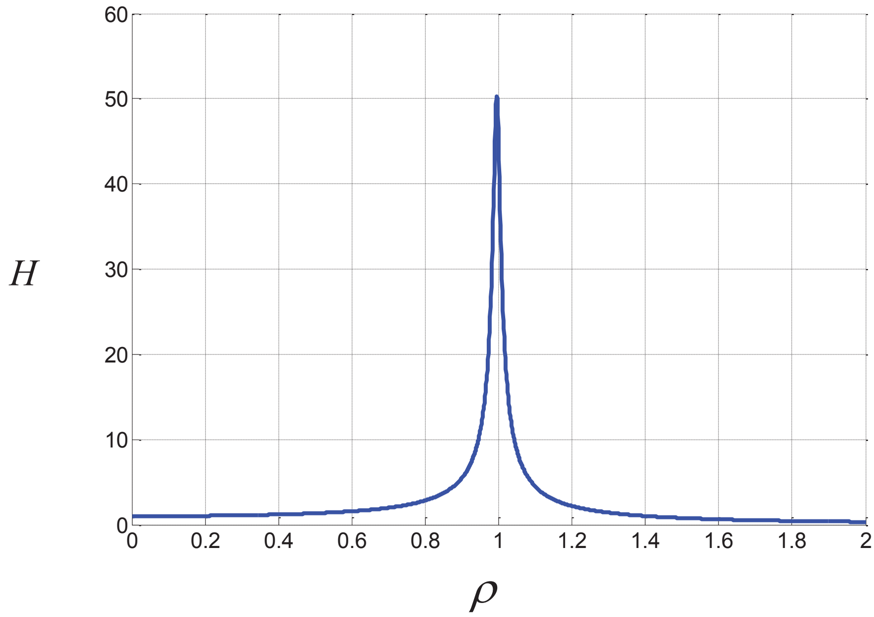
(5.16)

5.4 Case Study

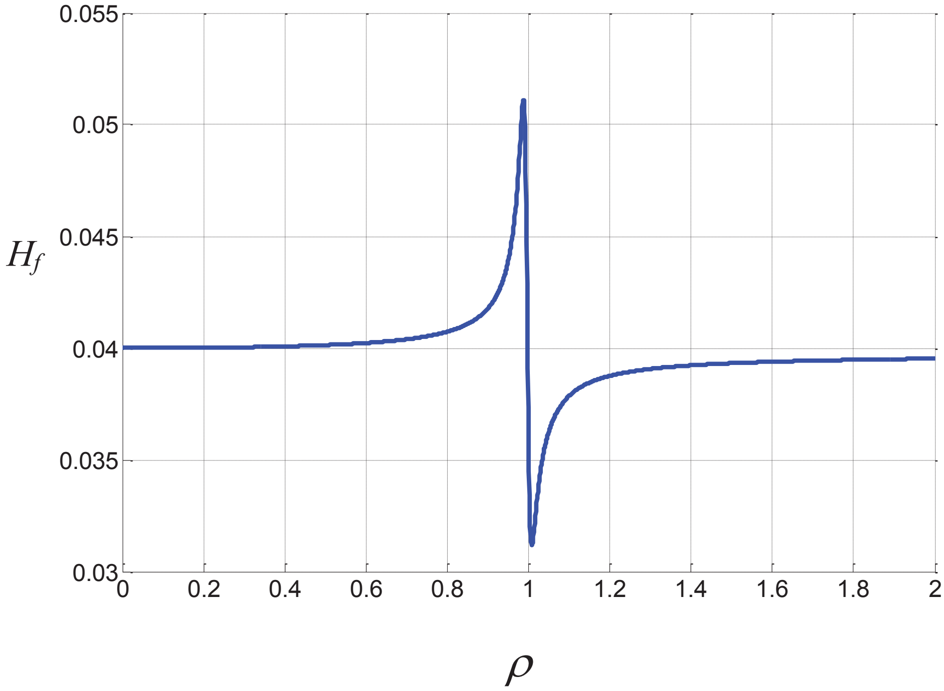
Tall building with conventional façade could be represented as a SDOF system although it may not be a precise model, but it could show the main performance of the structure. To illustrate the performance of the system, frequency response functions (also representing dynamic amplification factors here) are plotted with ρ values ranging from 0 to 2. The mass ratio between DSF and primary structure is assumed to be 1% and also DSF frequency to primary structure, frequency is assumed to be 50, 0.5 and 0.1, which represent the system from Conventional Façade to low stiffness connectors. In this study having a frequency ratio about 50 represents the system with rigid connector or conventional façade. For a damped single degree of freedom (SDOF) system subjected to harmonic load, the peak dynamic amplification factor could be obtained as follows:

$$H_{SDOF} = \frac{1}{2\xi\sqrt{1-\xi^2}} \tag{5.17}$$

In order to get the maximum dynamic amplification factor for single degree of freedom (SDOF) system Eq.5.17 can be used. The curve in Figure 5.2a is meant to represent a system with stiff connector which is representative of conventional façade. By comparing the result of Eq.5.17 and the peak in Figure 5.2b, dynamic amplification factor is less than 1 which means that there is no dynamic amplification for the DSF in this case.

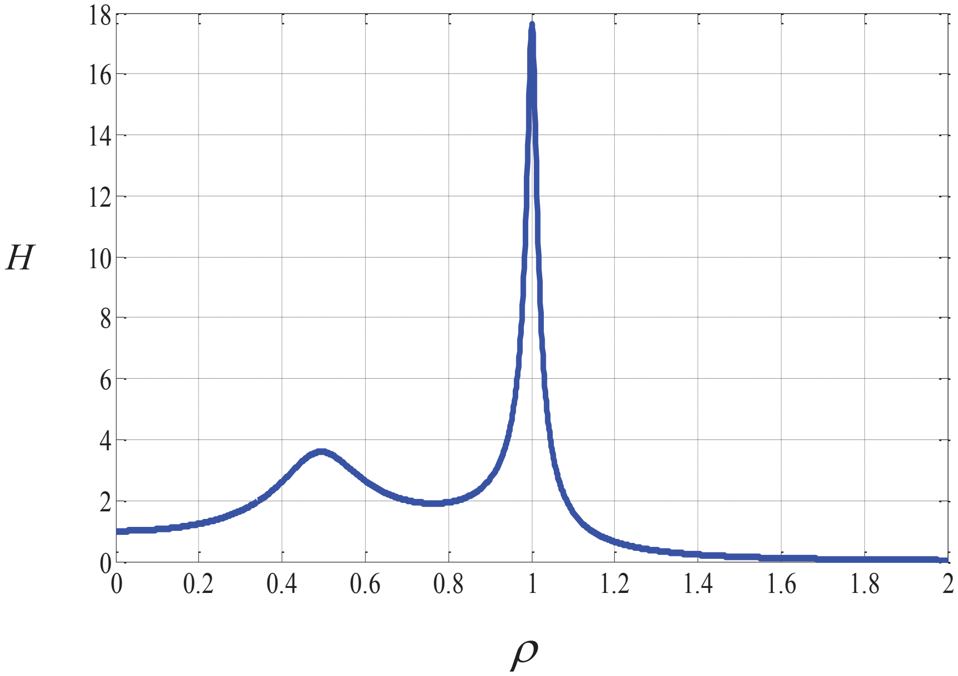


(a) H Plot

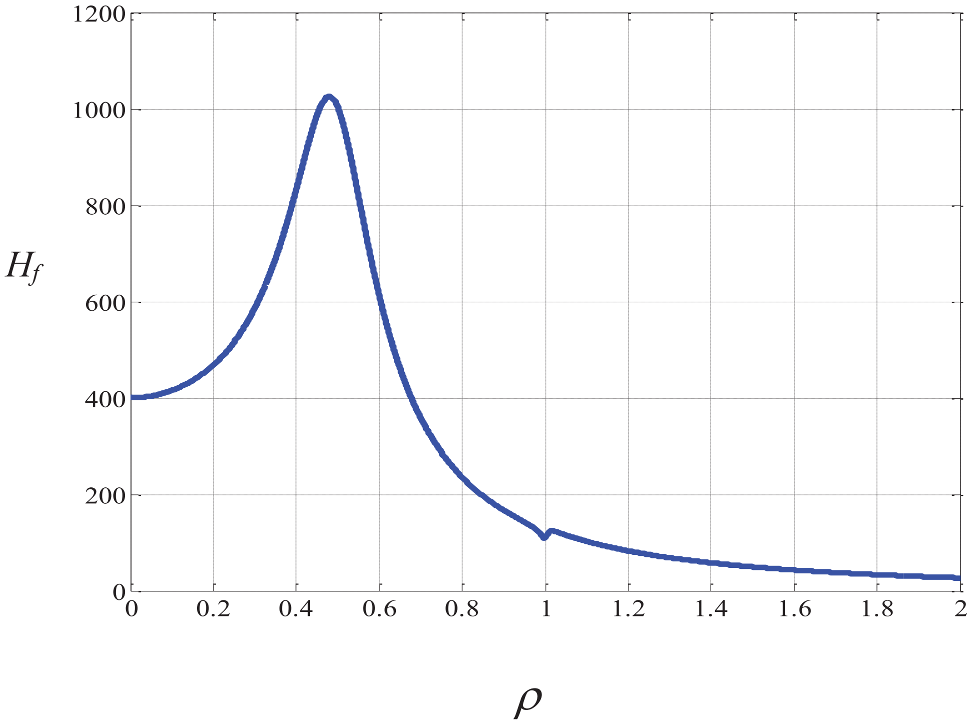


(b) H_f plot

Figure 5.2 Dynamic amplification factors for (a) the primary structure (H) and (b) DSF outer skin (Hf) with f (DSF outer skin frequency/primary structure frequency) =50



(a) H Plot



(b) H_f plot

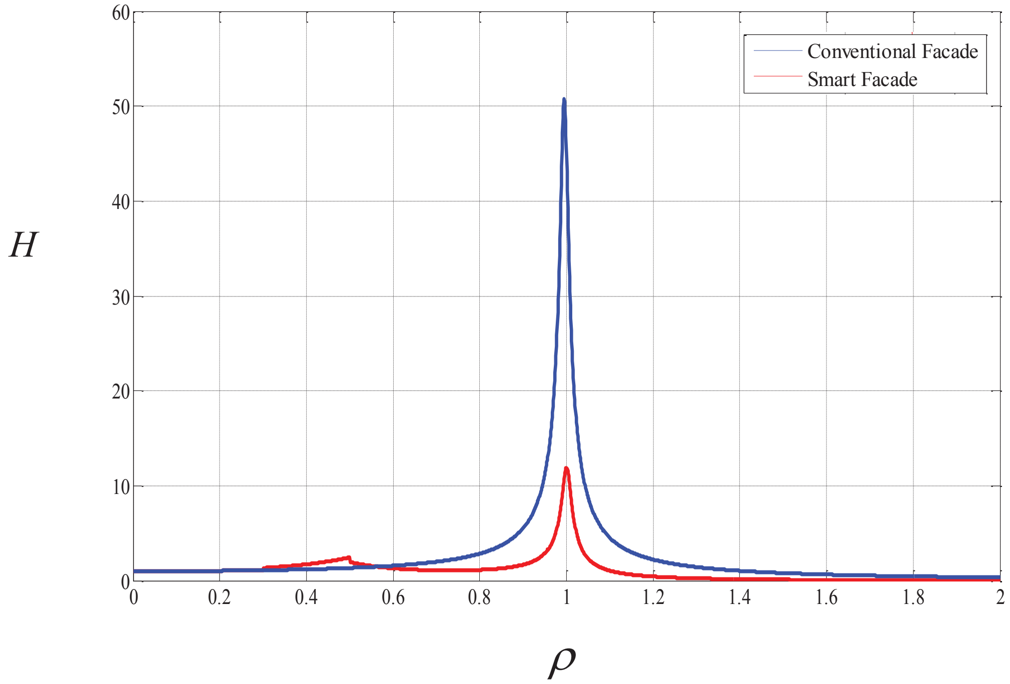
Figure 5.3 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) =0.5

Reducing the stiffness of connectors to the point where the DSF frequency has half the value of the primary structure, it leads to a noticeable reduction in the dynamic response of primary structure and as Figure 5.3a shows, the maximum H occurs when the forcing frequency is almost the same as the DSF connector frequency. With f=0.5, Figure 5.3b shows that the DSF dynamic amplification factor increases by about 1000 times with 20% damping. Compared to the conventional case without the proposed DSF system, the dynamic response of the primary structure is reduced by more than 64%. The above equations are obtained based on a linear system and constant values for stiffness and damping ratio. However, using a low frequency façade system could reduce the response of a structure, but it will also increase the relative displacement of façade panels. Changing the DSF connector stiffness corresponding to input load could help to control excessive movement of façade panels and maintain a similar reduction in response of structures.

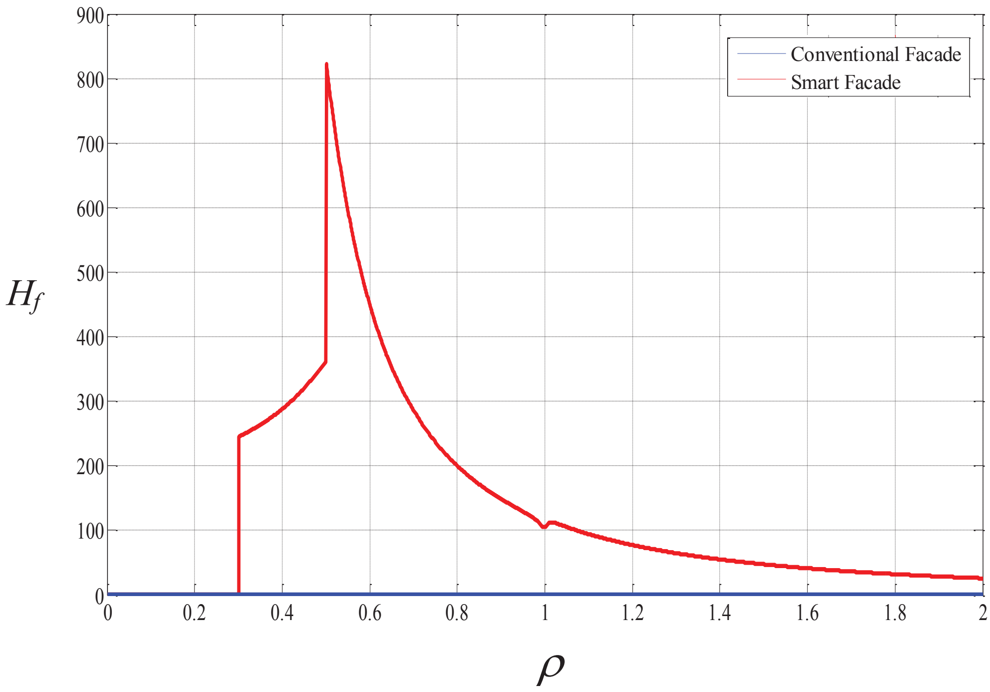
The following case represents the results of using different stiffness in connectors. As shown in previous results, having low stiffness connectors is critical to reduce the structural response by 50%. Maximum response of the structure occurs when the ratio of forcing frequency to primary structure, ρ, is equal to one. However, with variable f, this response, decreases by more than 50% and also, Figure 5.4b shows that the new arrangement of stiffness could be able to control the DSF dynamic amplification factor. Compared to the conventional case without the proposed DSF system, the dynamic response of the primary structure is reduced by more than 50% as mentioned earlier.

By contrast, in Figure 5.4 and 5.5; increasing the damping ratio, plays an important role in controlling the DSF outer skin frequency ratio, but also, increasing the damping ratio has the reverse effect on primary structural response. As shown in Figure 5.5b the maximum outer skin frequency is reduced noticeably which makes it more realistic for practical applications.

It is worth noting that for most, but not all structures, the response of the structure can be governed by more than just the fundamental mode of vibration and depending on where we add damping, in terms of mode number, the damping in other modes may increase or decrease depending on the energy contained in each mode. For a complex multi- mode system it is rare but not impossible to add damping to higher modes at the expense of reducing damping in the fundamental mode which is the mode of the main structure. This phenomenon requires further investigation to identify the exact mechanism responsible for this but is deemed beyond the scope of this thesis.

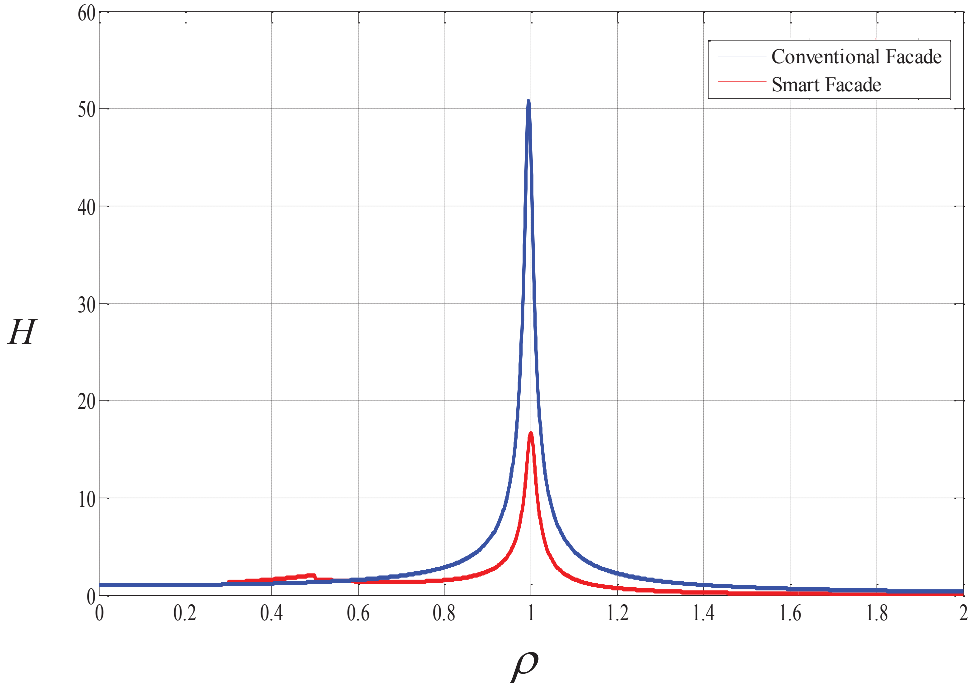


(a) H Plot

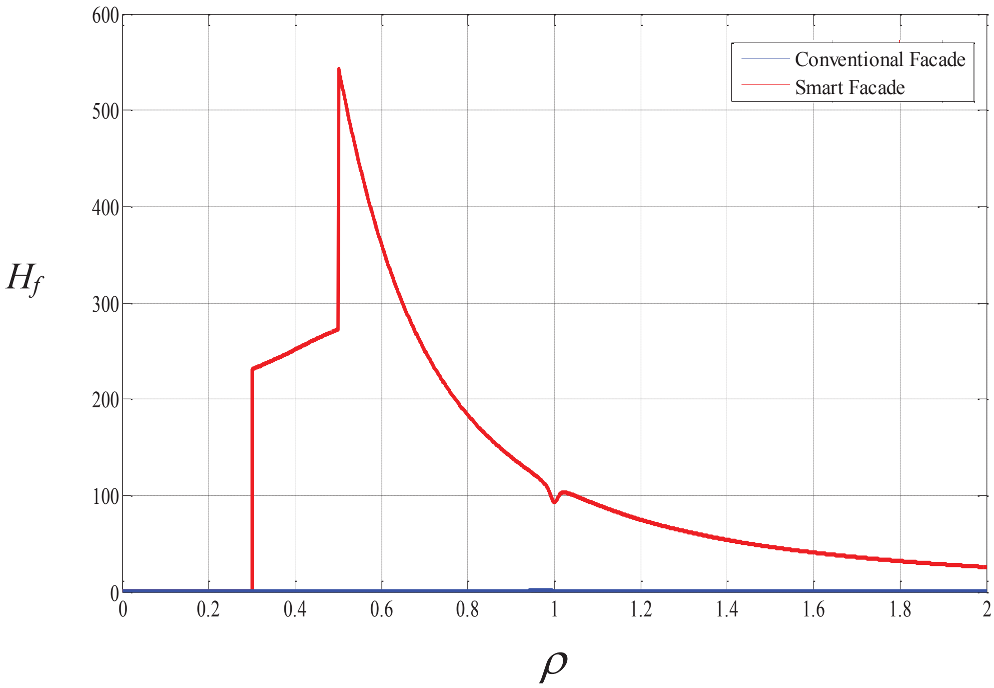


(b) H_f plot

Figure 5.4 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) = 0. 4 with 20% damping



(a) H Plot



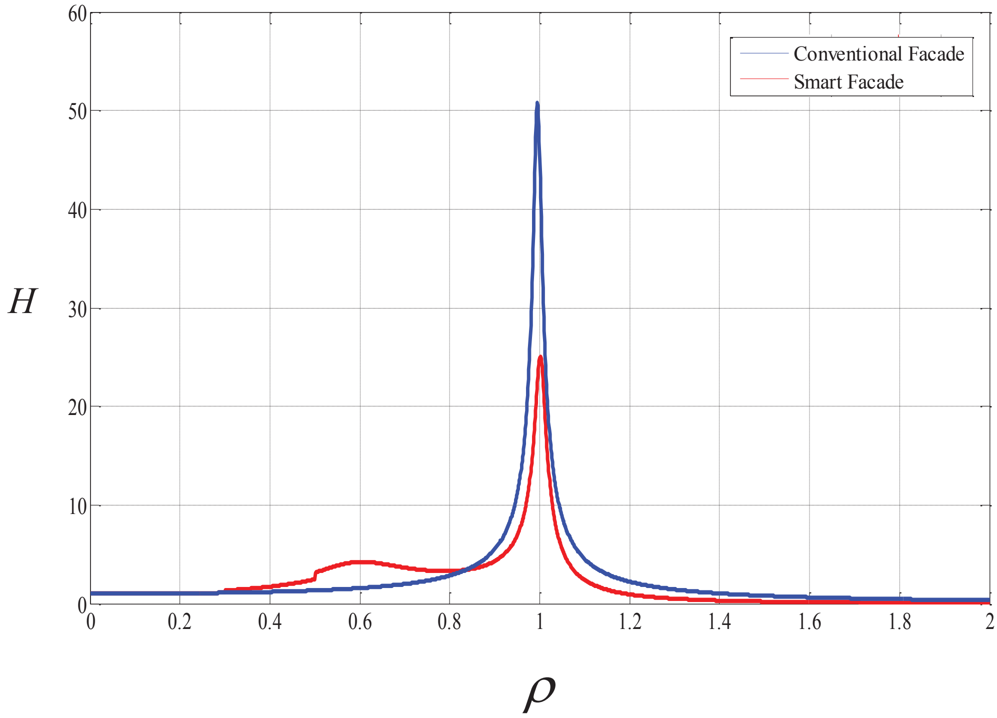
(b) H_f plot

Figure 5.5 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) =0.4 with 40% damping

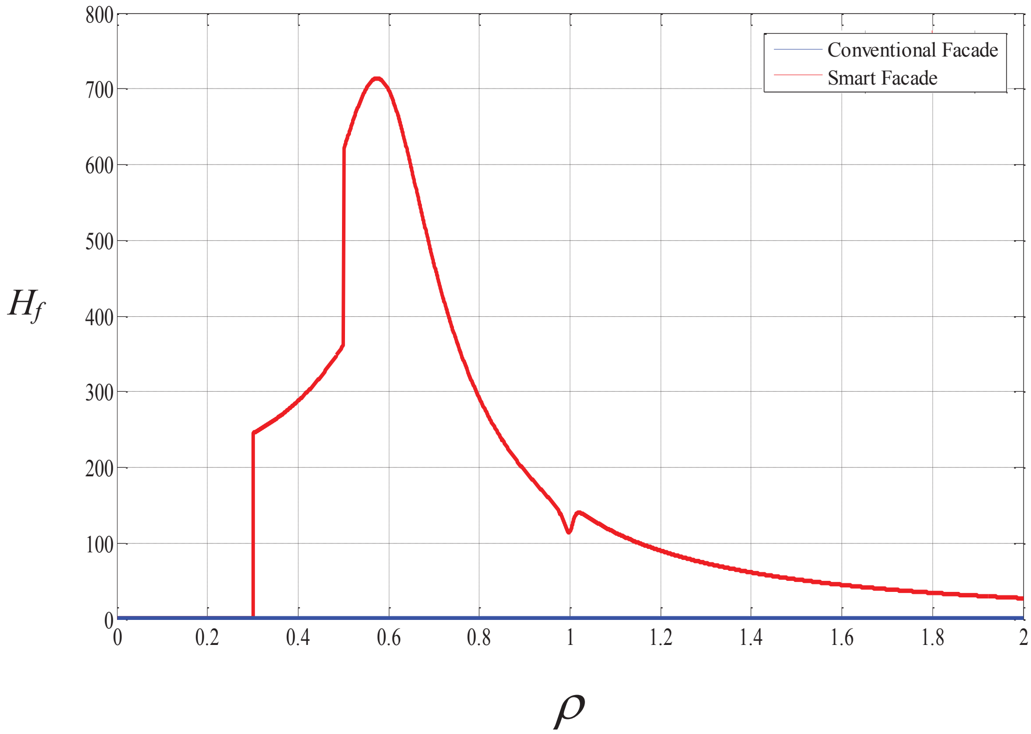
As stated earlier, decreasing the maximum ratio of outer skin frequency to primary structure frequency is the main aim with true potential for buildings. Significant frequency movement is occurring when the forcing frequency to primary structure frequency ratio is between 0.4 to

0.6. As can be seen from Figure 5.4b the ratio of DSF outer skin frequency is reduced where the DSF frequency has half the value of the primary structure frequency, however, H_f=800 is still potentially underestimated.

With minimum of f=0.6, Figure 5.6b shows that the DSF dynamic amplification factor increases by about 700 times with 20% damping. Compared to the conventional case without the proposed DSF system, the dynamic response of the primary structure is reduced by more than 37% (Figure 5.6a).



(a) H Plot



(b) H_f plot

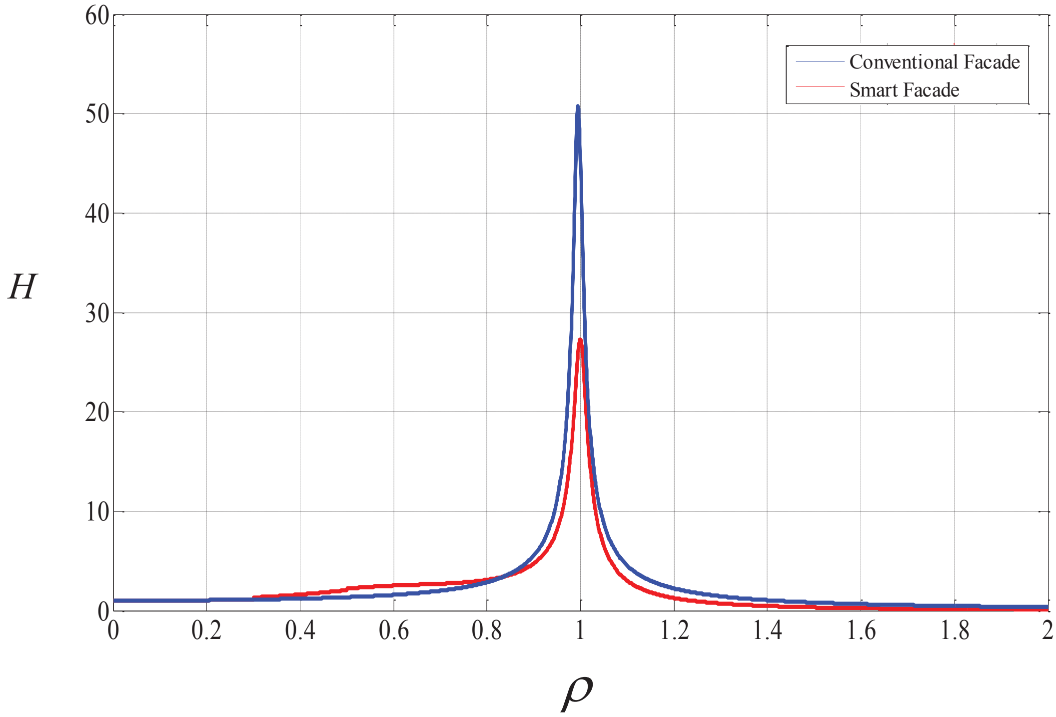
Figure 5.6 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) = 0.6 with 20% damping

According to what was stated earlier, damping has significant effect on the dynamic amplification factors for DSF outer skin. Regardless of primary structure, increasing the damping ratio is able to control the façade panel response. As Figure 5.7 shows by increasing the damping ratio by 20%, more or less the response of the primary structure has the same efficiency and also the dynamic response of panels is now limited to 400.

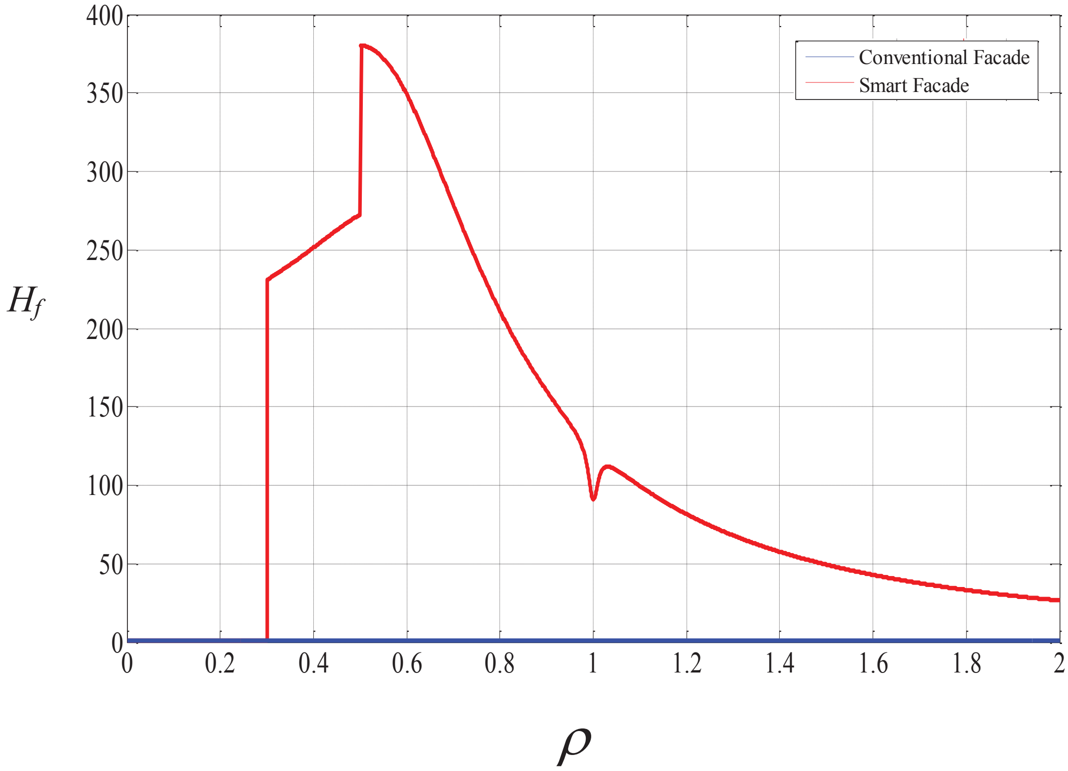
The case with f=0.7, as shown in Figure 5.8, simulates the scenario which has less beneficial effects compared to other above cases, but it still reduces, by around 30%, the response of the structure. In this case the peak of H value is close to 30 which is higher than before, but the dynamic response of the panels shows a value close to 500 which is the smallest value compared to other responses with 20% damping in connectors.

To adjust the scale accordingly to accommodate the changes of ratios between dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) i.e. reducing the dynamic amplification factor of the primary structure, is obtained by increasing the frequency ratio of DSF outer skin. Nevertheless, the curves in Figure 5.9 are meant to provide a clear view of this adjustment. As shown in Figure 5.9b, H_f is in the lowest value compared to previous results and also 35% reduction is achievable with this arrangement.

It is worth taking into consideration that there is an adjustment which leads to 35% reduction in structural response and also has the potential to be developed and built.

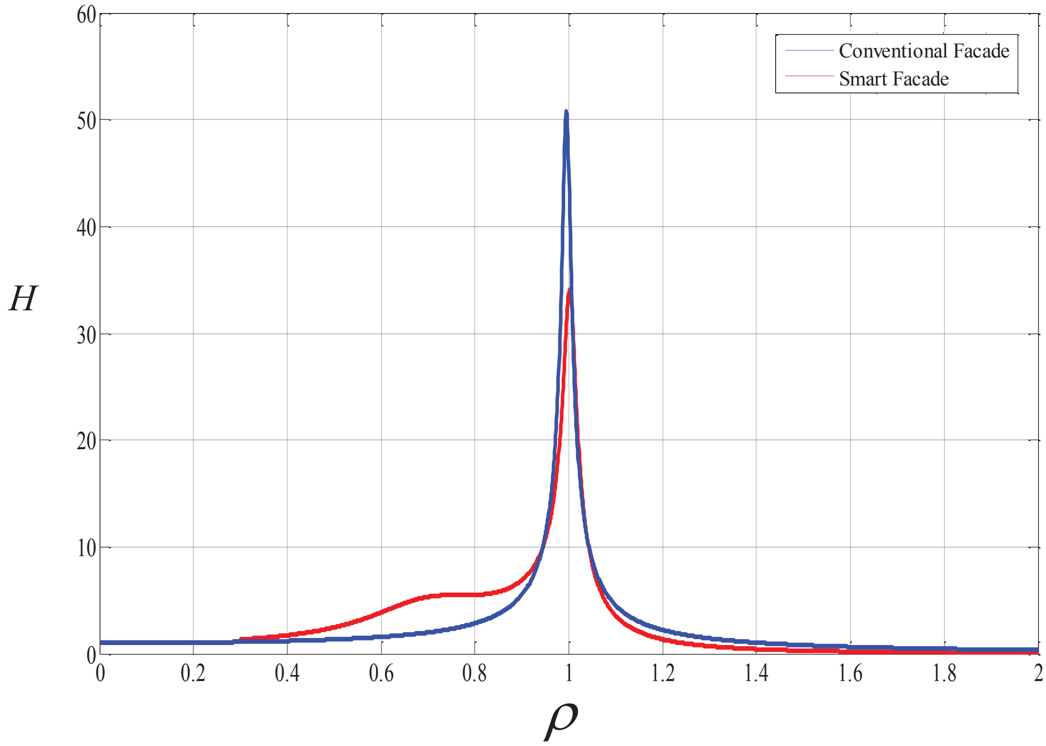


(a) H Plot

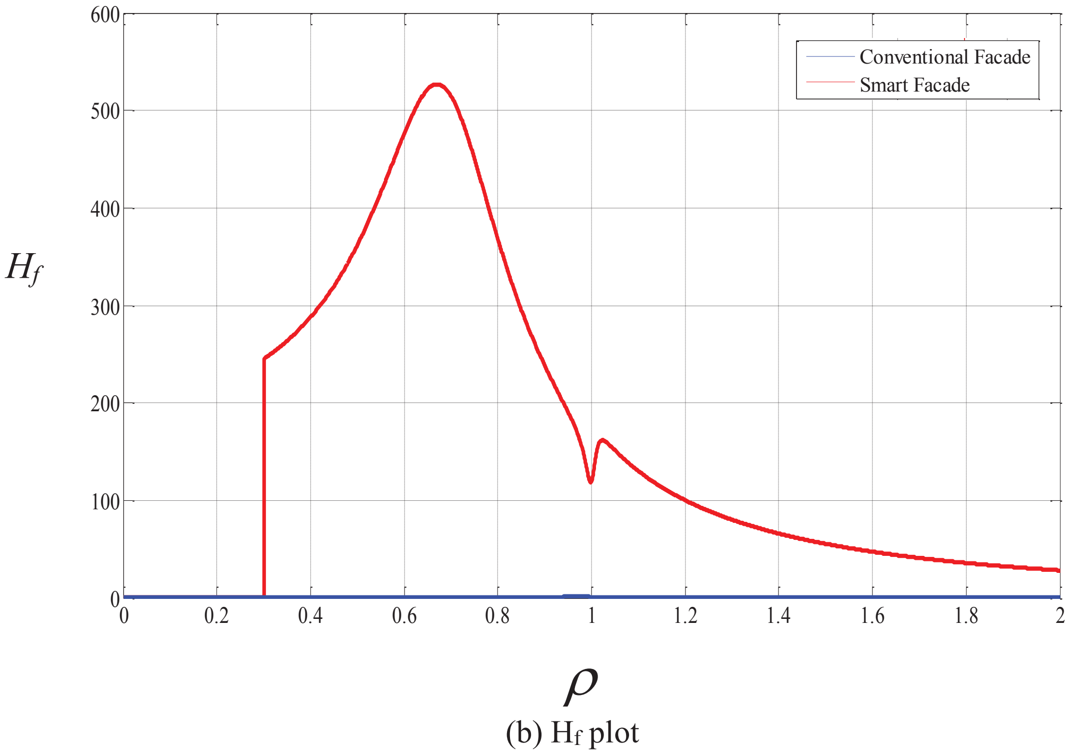


(b) H_f plot

Figure 5.7 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) =0.6 with 40% damping

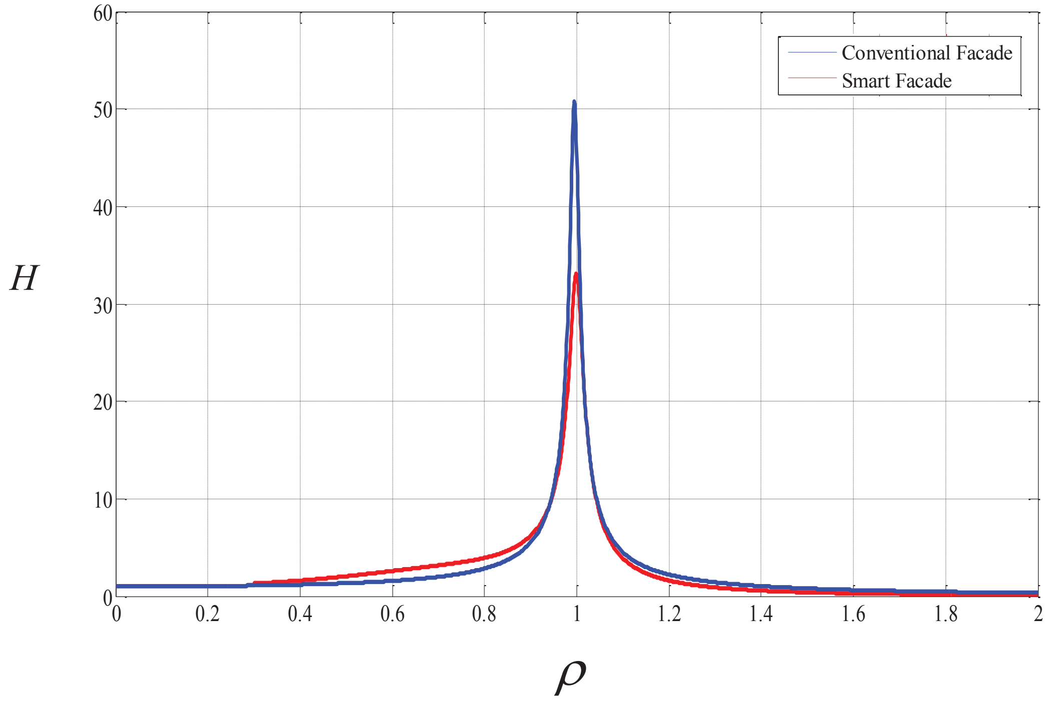


(a) H Plot

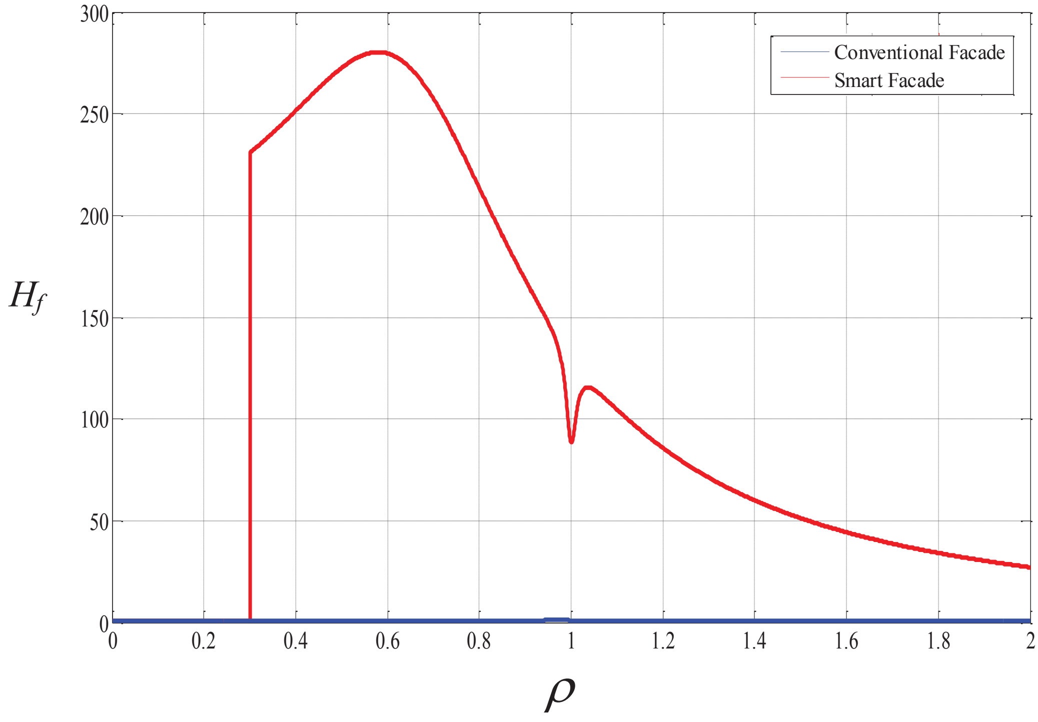


(b) H_f plot

Figure 5.8 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) =0.7 with 20% damping



(a) H Plot



(b) H_f plot

Figure 5.9 Dynamic amplification factors for the primary structure (H) and DSF outer skin (H_f) with f (DSF outer skin frequency/primary structure frequency) =0.7 with 40% damping

5.5 Conclusions

Double skin façade in tall buildings is one of the most advanced forms of façade systems available today. This study investigated the other potential functional benefits of double skin façades in tall buildings as lateral motion control devices. The results of this study show that using façade as a control system is feasible. Using the outer skin to filter input energy has significant effects on the response of the primary structure. Previous research shows that this system has potential to dissipate the wind energy, but requires a very large façade movement to do so which is not practical. This study represents a unique solution to make movable façade practical. By controlling the connector stiffness and introducing variable stiffness, one could reduce the primary structure response and also limit the movement of the outer skin of the façade to a practical value.

Chapter 6

6 Sensitivity Analysis

6.1 Introduction

Researchers recognize that results from simulation models are dependent, sometimes highly dependent, on values employed for critical variables. To account for this, analysts sometimes conduct sensitivity analyses with respect to key variables. Examples of key variables include behavioural parameters, input loadings, and shape of structures. As stated earlier, this novel system should perform in the way that can be proper and safe to install on the structures therefor, looking at different parameters and their effect is one of the important issues.

In this chapter, the key variables, including Behavioural parameters such as different stiffness slopes, damping ratio and the length of soft path behaviour are covered.

This chapter consists of series of sensitivity analyses, which contain a comparison between correlated and uncorrelated wind forces applied to the structure and also describes the performance differences pertaining to linear and multi linear behaviour of façade damper with respect to the displacement history of panels and number of crossings. It can also describe the effects of mass and stiffness ratio that is essential in terms of having different types of façade with different weights and also coupling between variable panel weight and stiffness ratio. The low stiffness part in the façade damper behaviour system makes the system to behave like a multi tuned mass damper. Further analyses is presented to show the optimum length for the second slope needed to ensure the effectiveness of the system. As stated earlier, in the acceleration response history of the system, spike phenomena occur due to sudden change of slope that has to be resolved by smoothing the transient part of the high stiffness ratio to low ratio. In the last part of this chapter, the response trend of a structure subjected to the range of wind load is presented in order to describe the gain value of the system.

6.2 Wind pressure coefficient

When the wind interacts with a building, both positive and negative (i.e., suction) pressures occur simultaneously (Figure 6.1). It is worth noting that negative pressures are less than ambient pressure, and positive pressures are greater than the ambient pressure. A building must have sufficient strength to resist the applied loads in order to prevent wind-induced building failure.

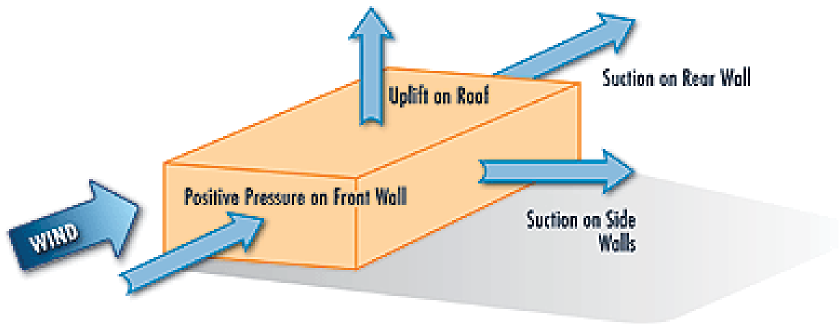


Figure 6.1 Schematic of wind-induced pressures on a building

Obtaining the design wind velocity is just the first step in determining the design wind loads on buildings. The second step is to determine the wind pressure coefficients that are applicable to the building. Wind pressures and loads on opposite sides of a building consist of windward and leeward components. The net wind pressure and hence load is the combination of the windward and leeward pressures making up the total load. The most typical values generally adopted for the pressure coefficients are +0.8 and -0.5 for windward and leeward faces of a rectangular shaped building, respectively.

6.3 Sensitivity Analysis on Stiffness effect

The idea of a façade acting as an energy absorber imerged a decade ago and researchers are working on this concept since then to make it a reality.

Sensitivity analyses are carried out on changes to stiffness value. Four different stiffness values are chosen to cover most probable and achievable range of stiffnesses. The first trend, referred to as linear damper behaviour occurs when the stiffness is equal to 2kN/mm which presents the conventional façade as shown in Figure 6.2 (red line). The second behaviour represents the 1kN/mm dampers that leads to improved performance by ensuring realistic movement values in façade panels. The proposed damper façade behaviour is shown by two relatively close stiffness.

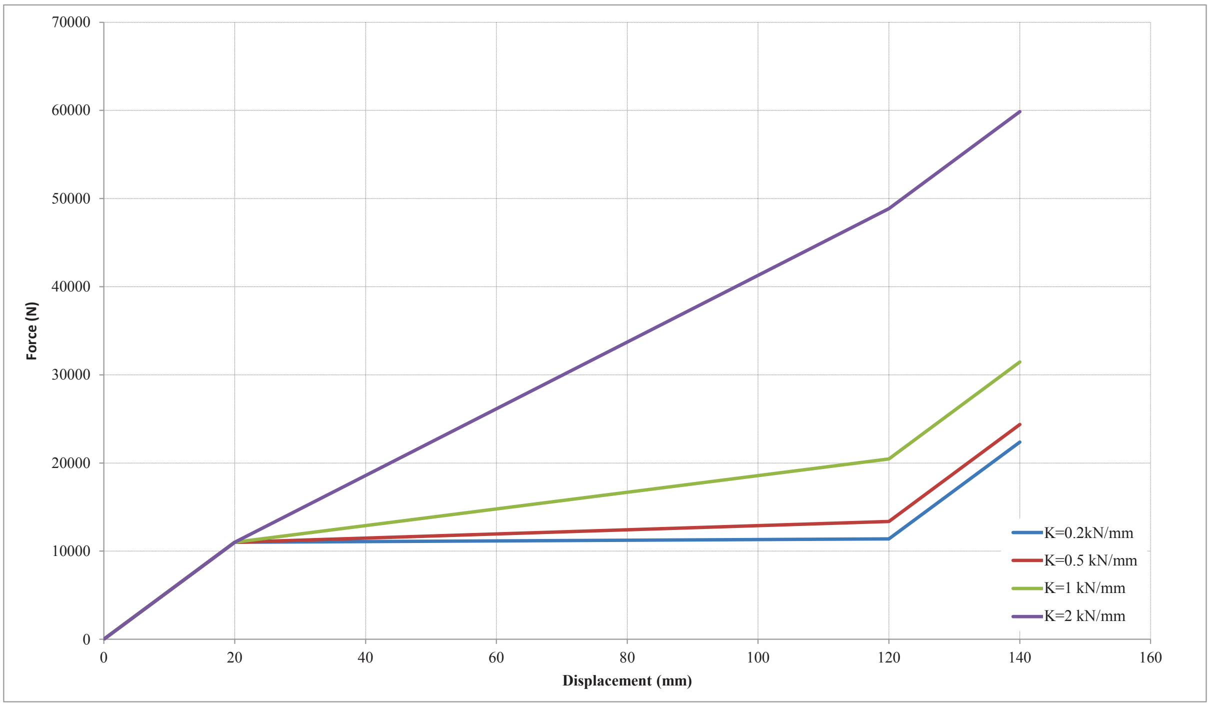


Figure 6.2 Comparing different stiffness values on damper behaviour

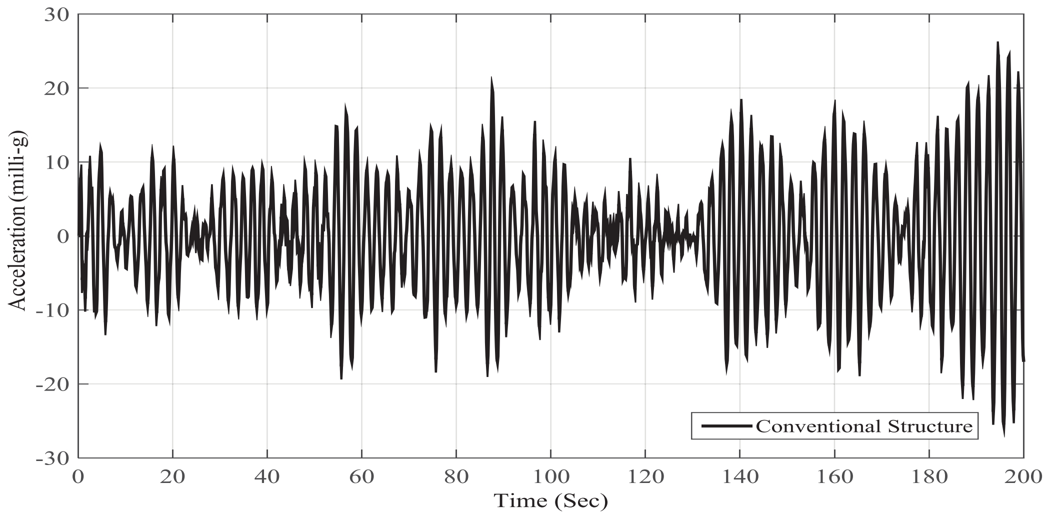


Figure 6.3 The acceleration response of the conventional building

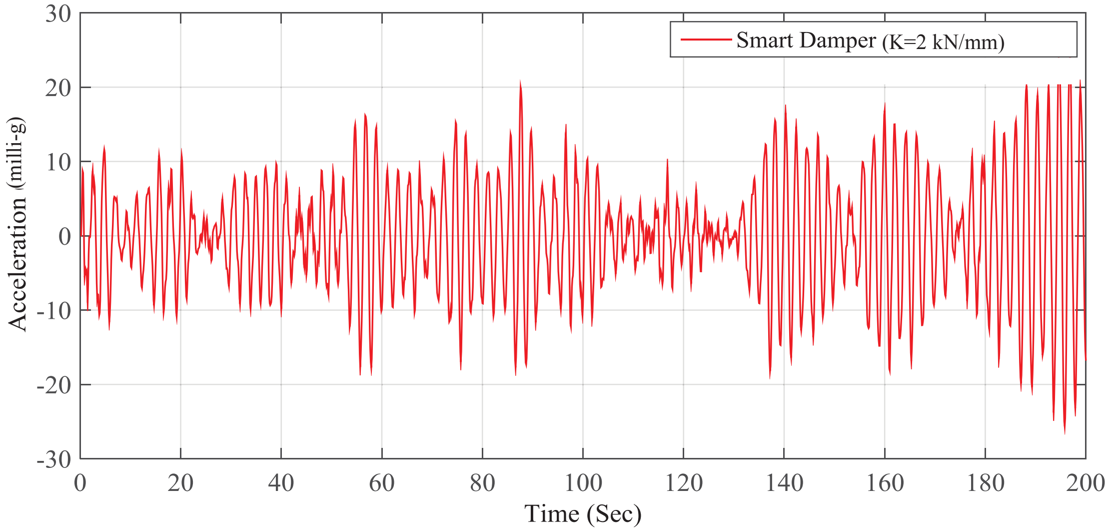


Figure 6.4 The acceleration response of the smart damper (K=2 kN/mm)

Figure 6.3 and 6.4 are meant to compare the response of conventional building with the response of the building which is equipped with smart damper façade with linear behaviour. The result shows that with 2 kN/mm stiffness value the response is the same as conventional structure, however in this case we have additional movement of the façade panels as well. Relative displacement of the façade panels (moveable to fixed) is one of the important parameters that has direct effects on occupant comfort. Figure 6.5 shows the relative displacement of façade panels. The response of damper with k=2 kN/mm is limited to 40 mm under the mean wind speed of 20m/s.

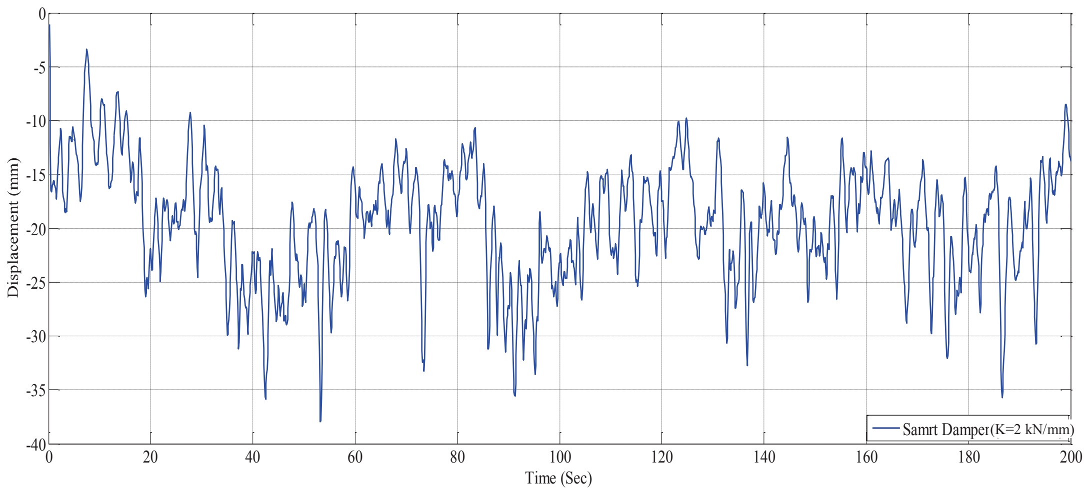


Figure 6.5 The relative displacement response of damper with K=2 kN/mm

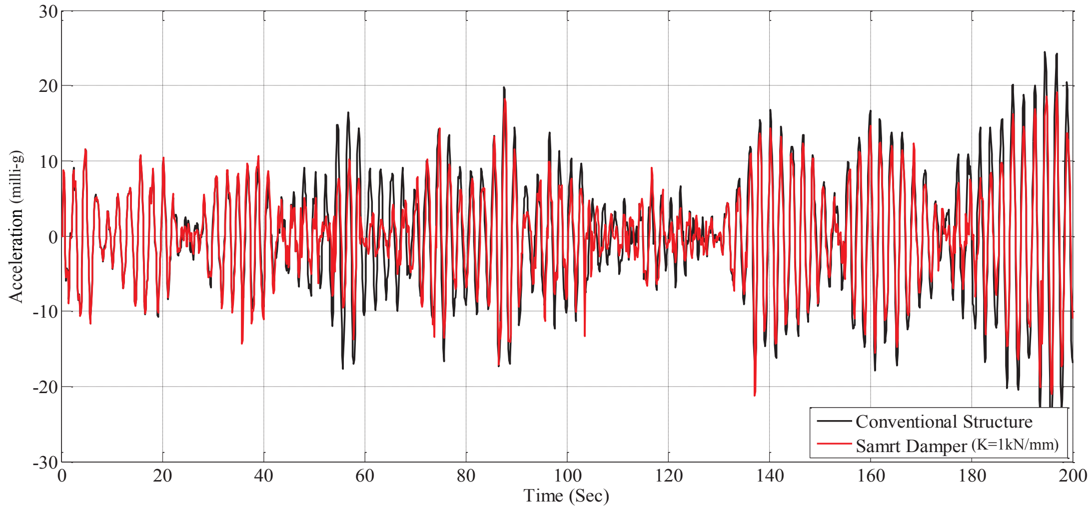


Figure 6.6 Comparing the acceleration response of the conventional building versus one with smart damper (K=1kN/mm)

By dropping the stiffness value to 1N/mm, which is half of the previous value, the system starts to reduce the top acceleration as shown in Figure 6.6. It can be demonstrated that the lower stiffness value leads the system to move at a certain frequency which could be the starting point of observing changes in the response of the buildings. The corresponding trend of damper displacement is shown in Figure 6.7 where the panels are allowed to move by up to 90mm.

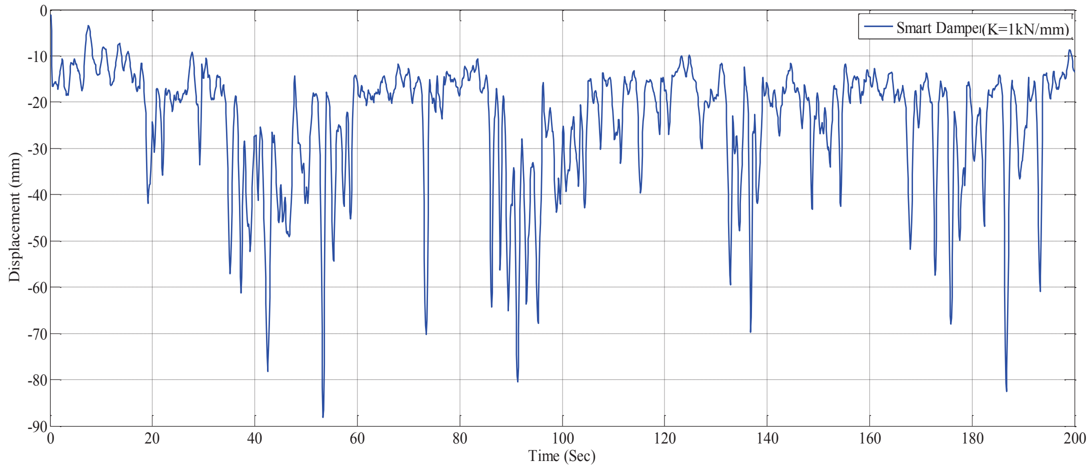


Figure 6.7 The relative displacement response of damper with K=1 kN/mm

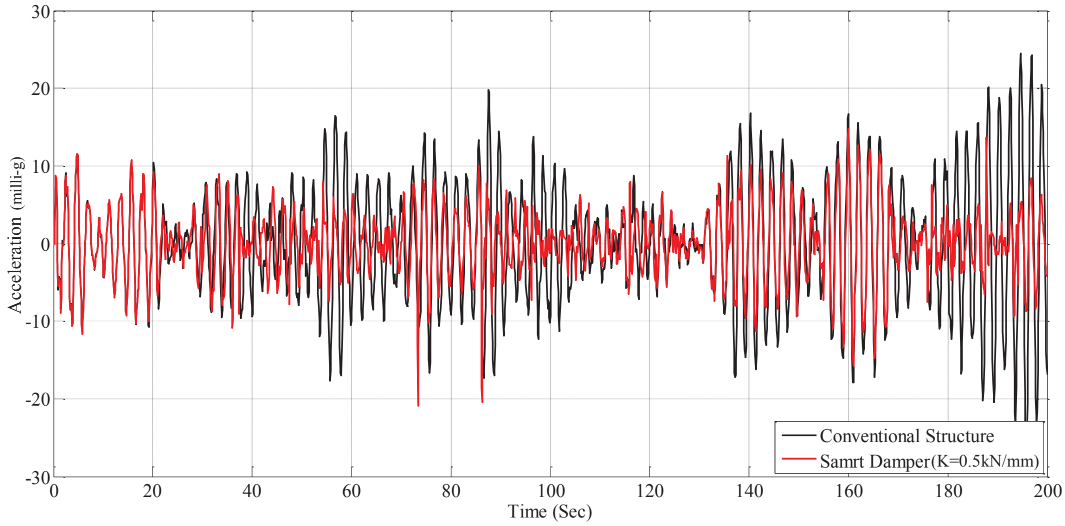


Figure 6.8 Comparing the acceleration response of the conventional building versus one with smart damper (K=0.5 kN/mm)

Three façade dampers with linear behaviour can lead to a noticeable reduction in top acceleration response of structures (Figure 6.8). As stated earlier, the smart system has beneficial effects when the façade panel assumes the second slope (soft slope) in a certain range. Panels fluctuate by the pattern assigned to the damper behaviour. By contrast, in Figure 6.7 and 6.9 it can be seen that the system starts to work when the panels start to move with a certain frequency.

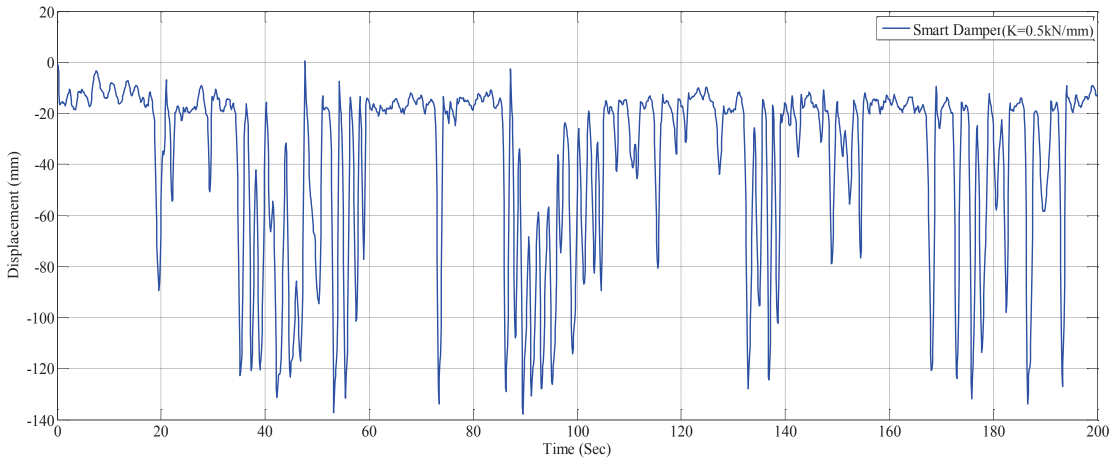


Figure 6.9 The relative displacement response of damper with K = 0.5 kN/mm

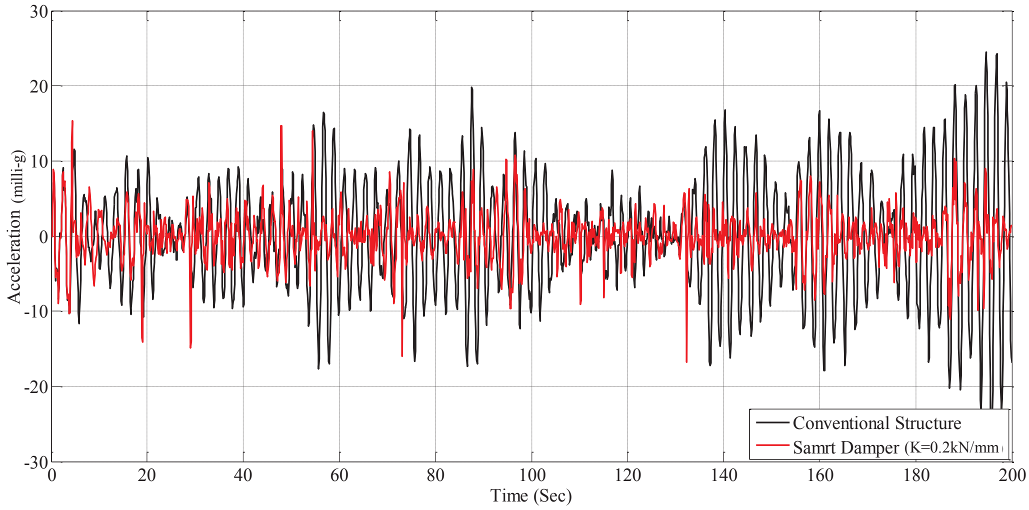


Figure 6.10 Comparing the response of the conventional structure versus one with smart damper (K=0.2kN/mm)

In order to increase the beneficial effect of the damper façade, an optimisation process is required. Remaining in the second slope is the key which leads to an optimum combination. Moreover, it is important to make the panels move in a proper frequency range. Although increasing the difference between slope ratios makes the damper harder to build, but it leads to a better performance (Figure 6.10).

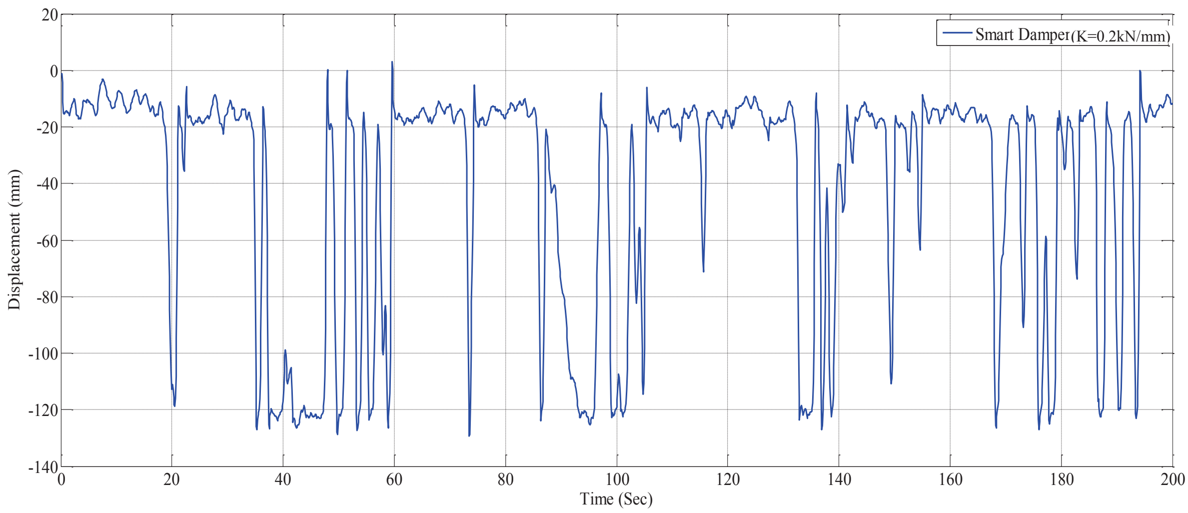


Figure 6.11 The relative displacement response of damper with K=0.2 kN/mm

Figure 6.11 is presented to demonstrate that the relative displacement of panels and crossing to the second slope could render this system more effective. Nevertheless, the curves show that oscillations around the second slope and remaining there make significant improvements in response of the structures.

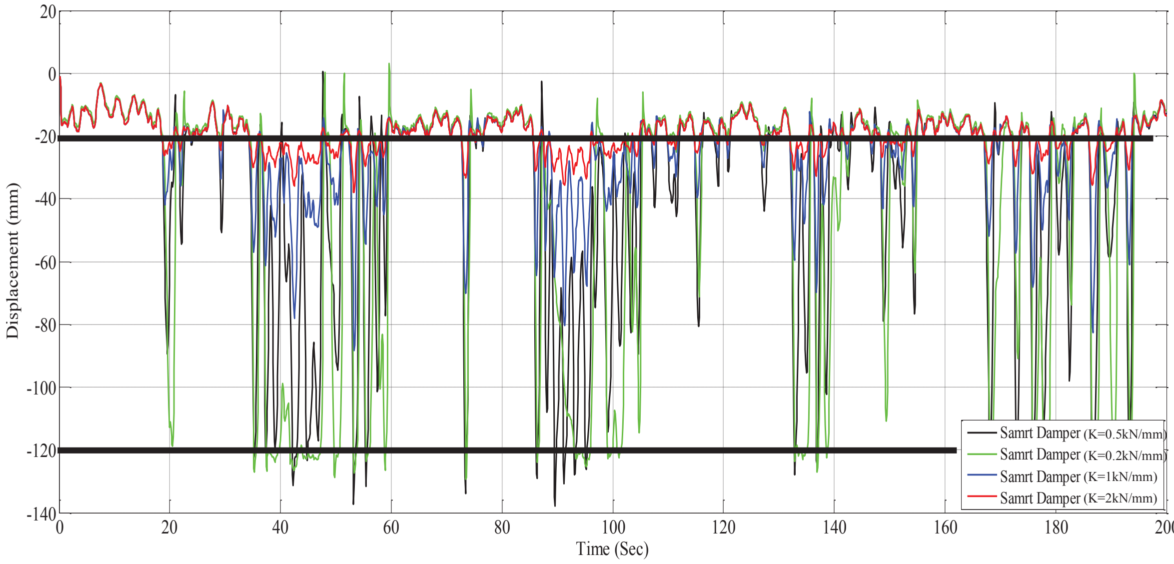


Figure 6.12 Comparing the displacement responses of dampers

As shown in Figure 6.12, we can divide the motion pattern into three phases. In order to ensure that façade panel does not move more than 20 mm under daily wind loads, the high stiffness ratio is chosen in the first phase. The second phase is meant to reduce the structural response which is achieved by the lower stiffness ratio. Due to lower stiffness ratio, panel vibrations lead to panels moving up to 100 mm. It is worth noting that excessive movement of panels is controlled by the third phase, which is associated with a higher stiffness ratio to ensure façade remains at a safe distance from the main structure.

6.4 Sensitivity Analyses Based on Mass and Stiffness Ratio

As stated earlier the number and the frequency of crossing a certain displacement threshold are the two key concepts which lead to having 50% reduction in both displacement and acceleration of the structure's response. Based on those key concepts, sensitivity analyses are needed to show how changes to these two parameters exploit the beneficial effects of the smart system. As both are affected by the stiffness ratio between the first slope and the second slope, it is critical to explore the extent of sensitivity of this system.

The weight of façade panels is the first variable in this sensitivity analysis, which is changed from 100kg per panel to 1000kg per panel. The second parameter which leads to having beneficial effects in this smart system is the stiffness ratio that plays the most important role. This is assumed to be in the range of 0.1 to 2 and has to be practical and definable in reality. As the first graph in this section, Figure 6.13 presents the performance of the smart system by assuming panel weights of 100kg each and 0.1 kN/mm as connector stiffness.

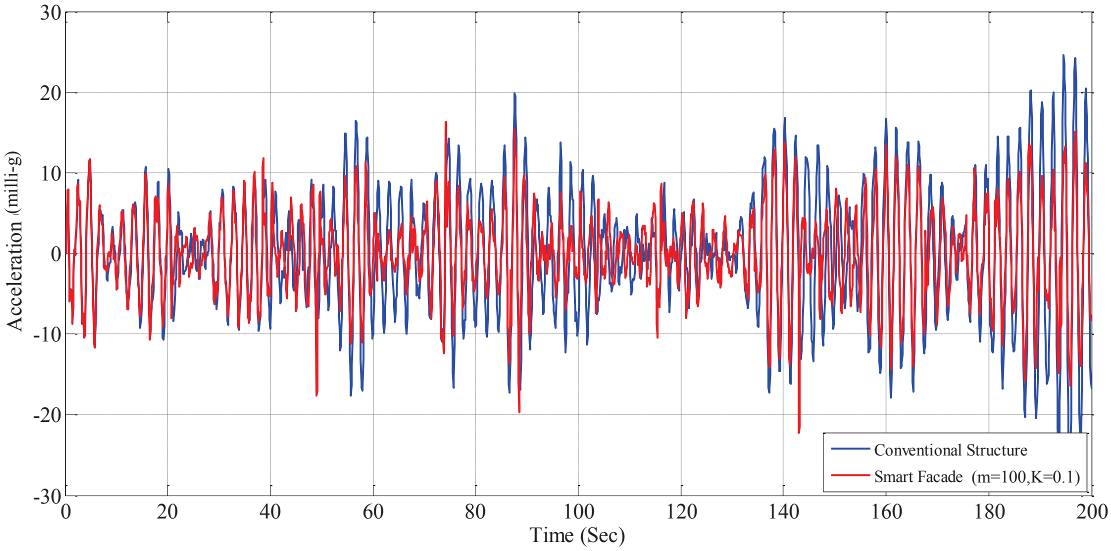


Figure 6.13 Performance of the conventional structure versus Structure equipped with smart façade (m=100, K=0. 1 kN/mm)

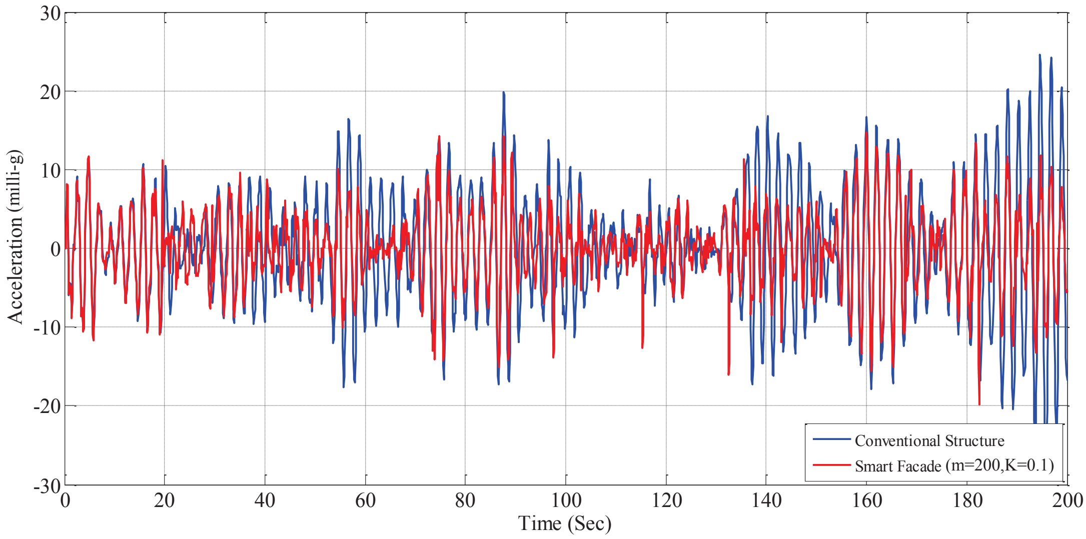


Figure 6.14 Performance of the conventional structure versus structure equipped with smart façade (m=200, K=0. 1kN/mm)

Increasing the weight of each panel to 200kg and maintaining the lowest stiffness ratio leads to marginal improvement in terms of top acceleration response. Figure 6.14 shows the structure equipped with smart façade having similar performance comparable to the previous one with lighter facade panels. Figure 6.15 presents the difference in response when the smart structures have different panel weights.

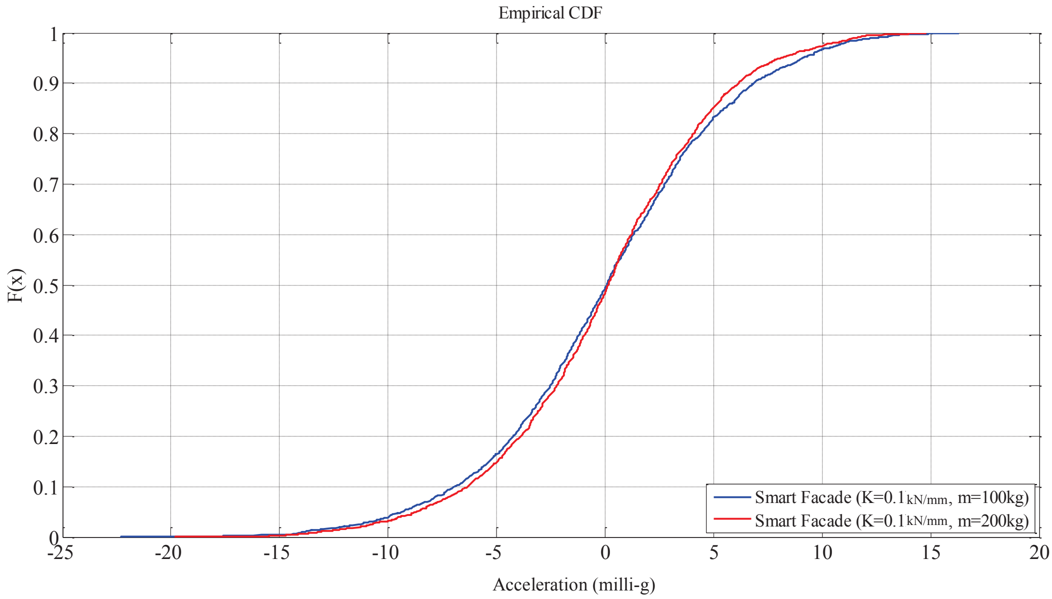


Figure 6.15 Cumulative density function of smart façade response, assuming 100kg vs 200kg per panel weight

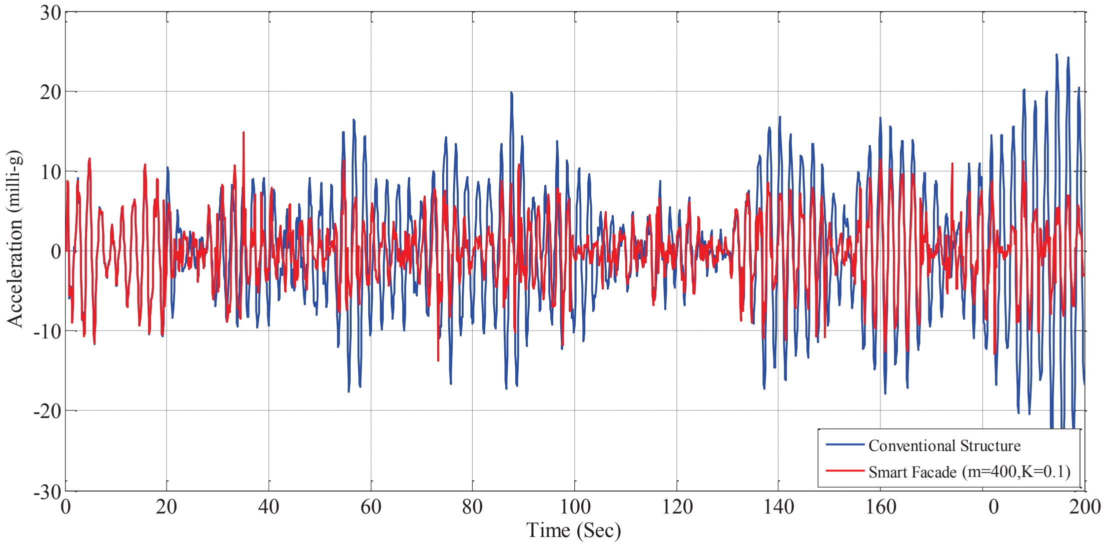


Figure 6.16 Performance of the conventional structure versus structure equipped with smart façade (m=400, K=0.1kN/mm)

The typical weight of facade panels used in industry is close to 400kg per panel and, therefore, using 400kg is the most appropriate weight for analysis. The performance of the structure with 400kg façade panels is presented in Figure 6.16 and Figure 6.17 displaying the performance trend of structure with different panel weights.

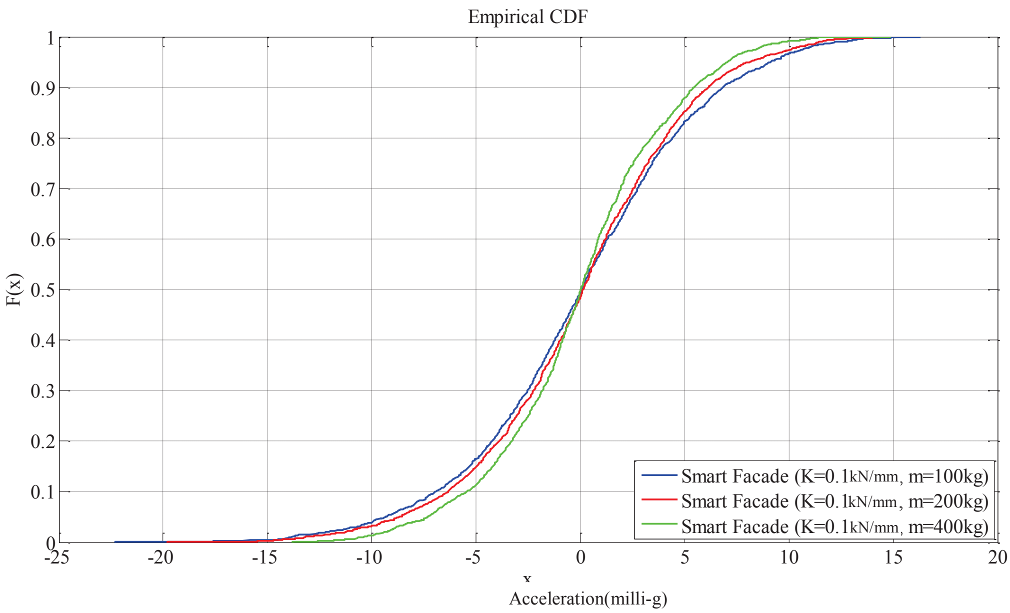


Figure 6.17 Cumulative density function of smart façade response, assuming 100kg, 200kg and 400kg per panel weight

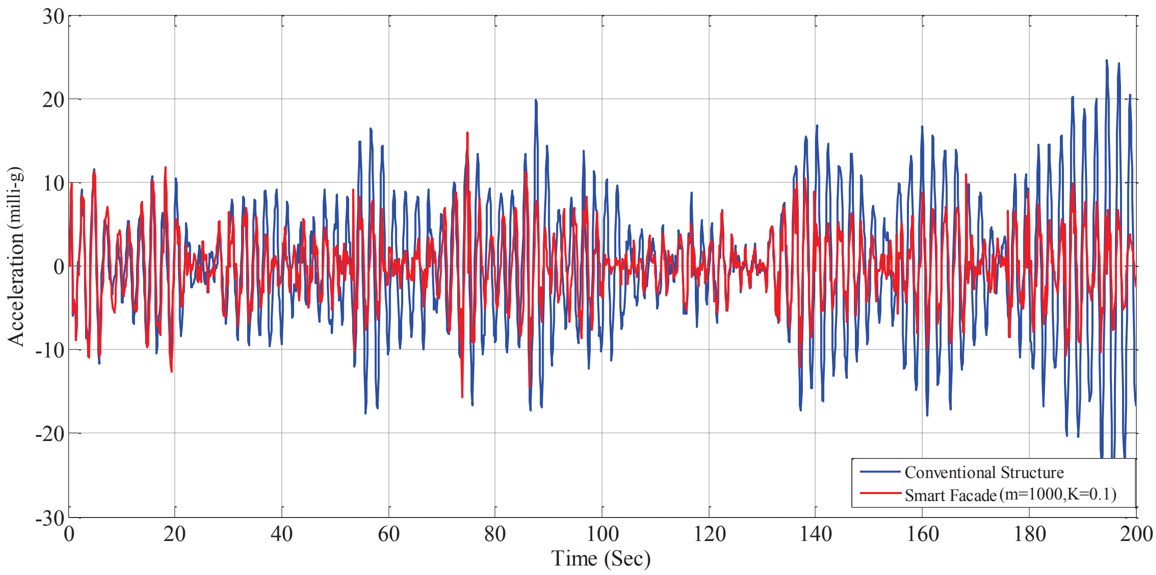


Figure 6.18 Performance of the conventional structure versus structure equipped with smart façade (m=1000, K=0.1kN/mm)

It is anticipated that increasing the weight of the façade should lead to better performance of the smart system (Figure 6.18). However this is not necessarily the case and the performance of the system is rather insensitive to weight, for example it is seen that increasing the weight of each panel by a factor of two does not lead to a significant difference in the performance of the structures (Figure 6.19).

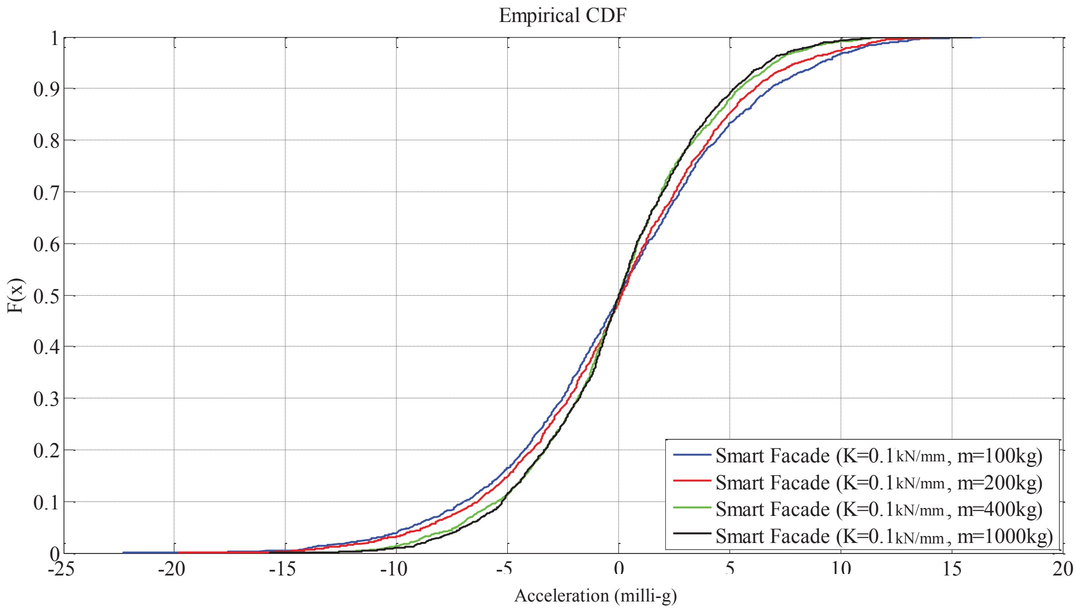


Figure 6.19 Cumulative density function of smart façade response, assuming 100kg, 200kg and 400kg and 1000kg per panel weight

Table 6.1 presents a summary of the first sensitivity analysis. The response of structure equipped with different smart damper is shown and standard deviation of each case is also presented.

Table 6.1 Standard deviation of the response of structure equipped with different facade damper

	Conventional	K=0.1kN/mm m=100kg	K=0.1kN/mm m=200kg	K=0.1kN/mm m=400kg	K=0.1kN/mm m=1000kg
Acceleration (milli-g)	7.697	5.454	5.012	4.235	4.085
Displacement (mm)	9.41	6.54	6.15	5.25	5.32

Based on these results it can be concluded that increasing the weight of the panel up to a certain point is helpful but it could also make the response slightly worse. In columns 5 and 6 in Table 6.1, it can be seen that the performance of the system is not propoetional to the panel weight.

The following, Figures and tables present the effect of the higher stiffness ratio and show the effect of the weight of the panels in this range. Figure 6.20 compares the performance of the conventional structure (blue line) with smart structure façade (red line). In this Figure each façade panel weight is 100kg and the stiffness value is 0.2kN/mm for the soft part.

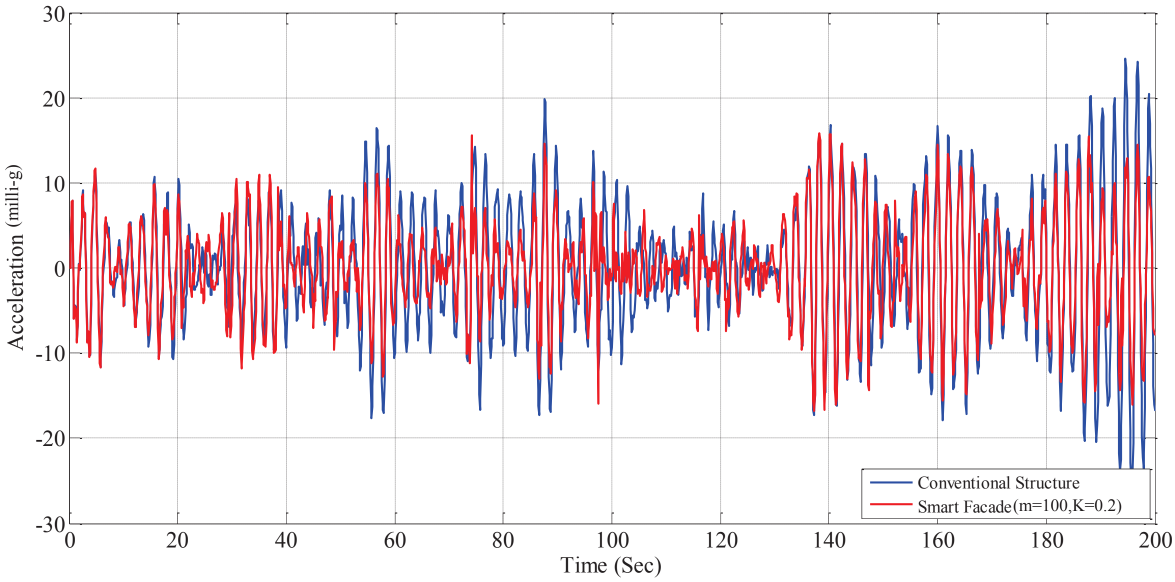


Figure 6.20 Performance of the Conventional Structure versus Structure Equipped with Smart Façade (m=100, K=0.2kN/mm)

The curves in Figure 6.21 show the performance comparison between damper façade with k=0.1kN/mm and k=0. 2kN/mm. It is expected that increasing the soft stiffness to 0.2kN/mm will lead to a small change in terms of acceleration response. It can be noted that the combination of 100kg panel and 0.2kN/mm stiffness display a better performance when the top acceleration of the structure exceeds from 5mg.

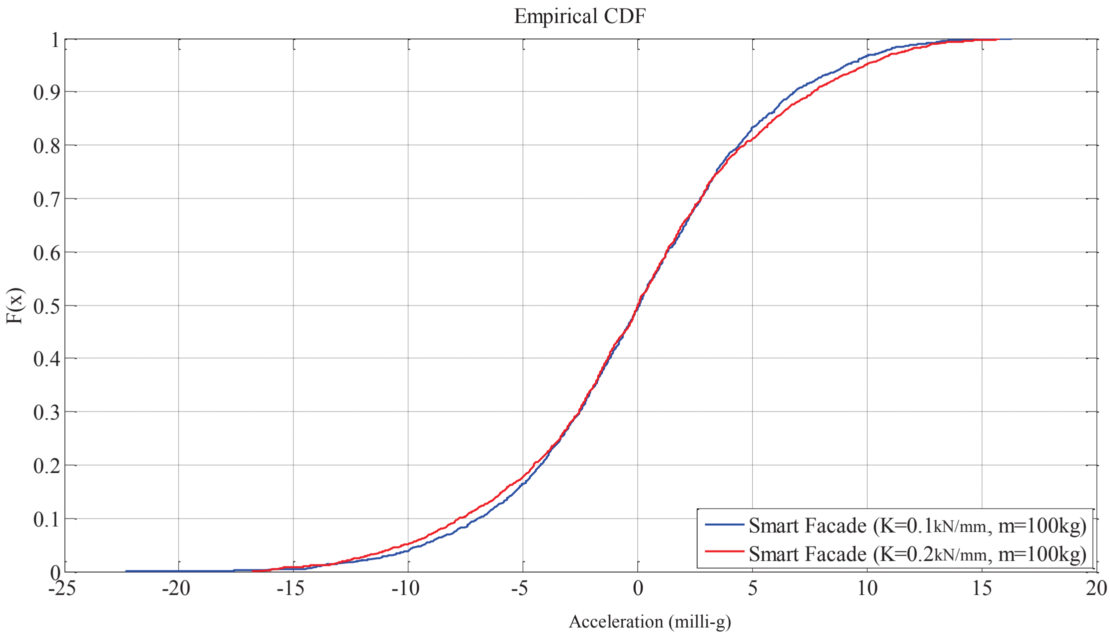


Figure 6.21 Cumulative density function of smart façade response, considering K=0.1kN/mm and 0.2kN/mm with 100kg per panel weight

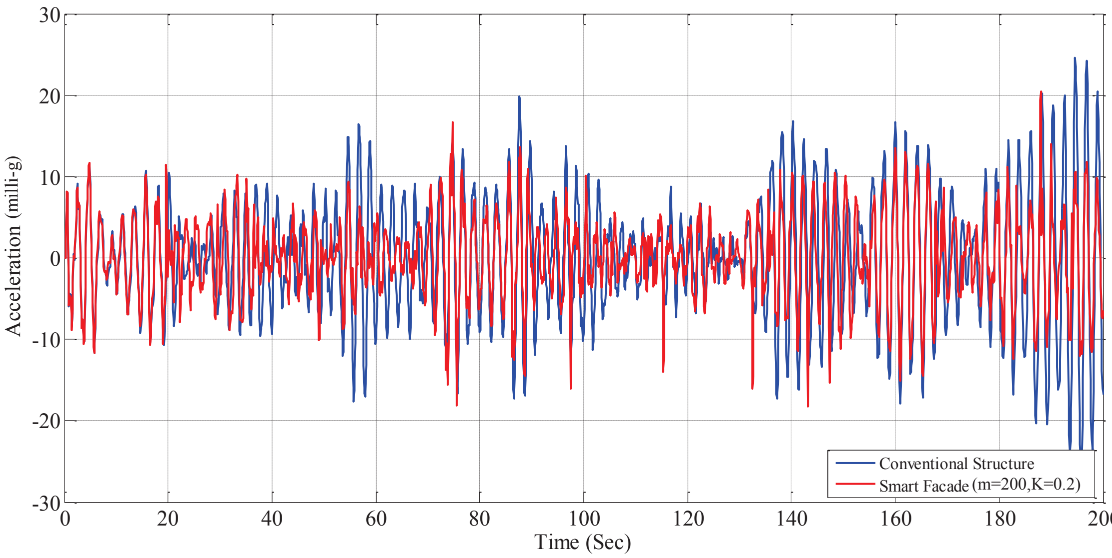


Figure 6.22 Performance of the conventional structure versus structure equipped with smart façade (m=200, K=0.2kN/mm)

Using 200kg facade panel and choosing the stiffness as 0.2kN/mm helped the system to show a beneficial effect in terms of top acceleration response. Figure 6.22 shows the structure equipped with smart façade having a better performance compared to previous damper combinations with lighter facade panels. Figure 6.23 presents the difference in response when the smart structure has different panel weights of 100kg and 200kg, relatively.

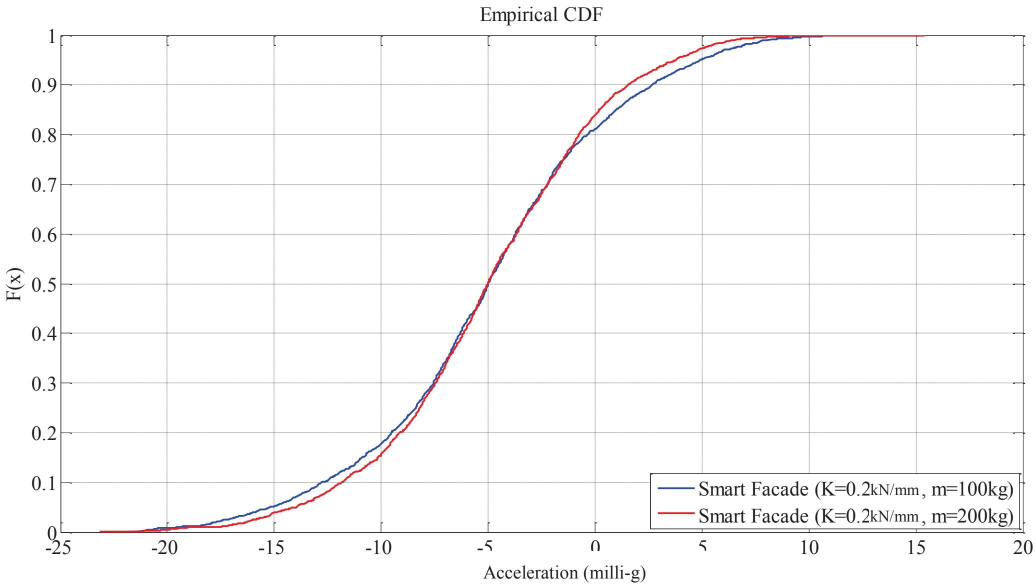


Figure 6.23 Cumulative density function of smart façade response, assuming 100kg, 200kg per panel weight

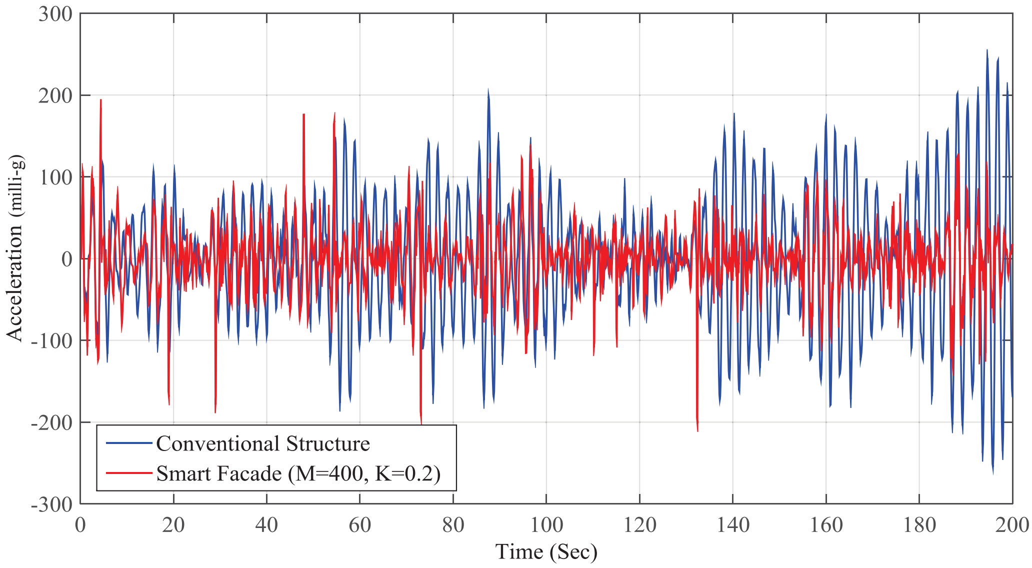


Figure 6.24 Performance of the conventional structure versus structure equipped with smart façade (m=400, K=0.2kN/mm)

The best frequency which leads to highest reduction is obtainable by using 400kg panels and 0.2kN/mm stiffness. Figure 6.24 demonstrates the performance of the structure by comparing the conventional structure response with smart structure. This is clearly seen in Figure 6.25 in which the green line is displaying the best performance of the structure so far which happens due to the right frequency combining with the filtering effect of smart dampers.

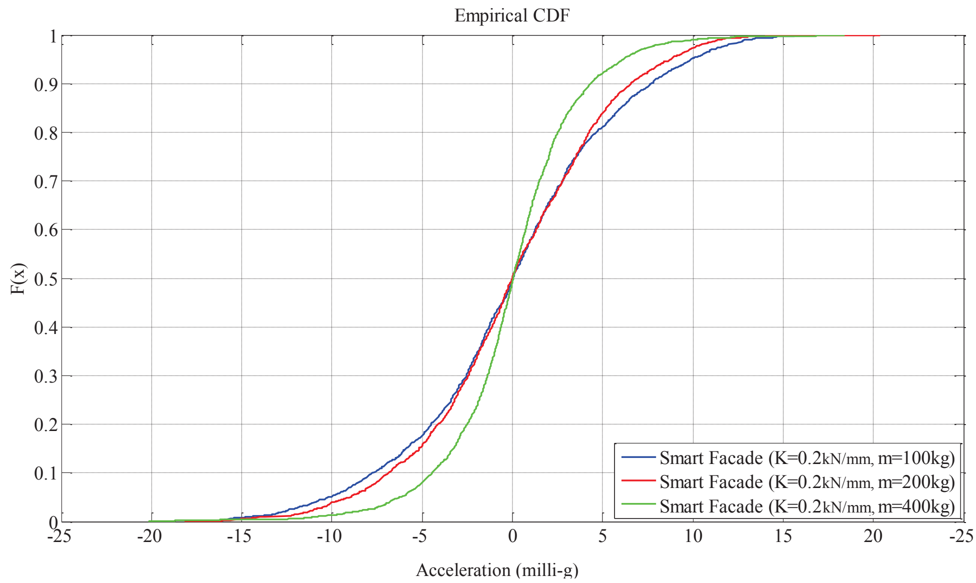


Figure 6.25 Cumulative density function of smart façade response, for 100kg, 200kg and 400kg per panel weight

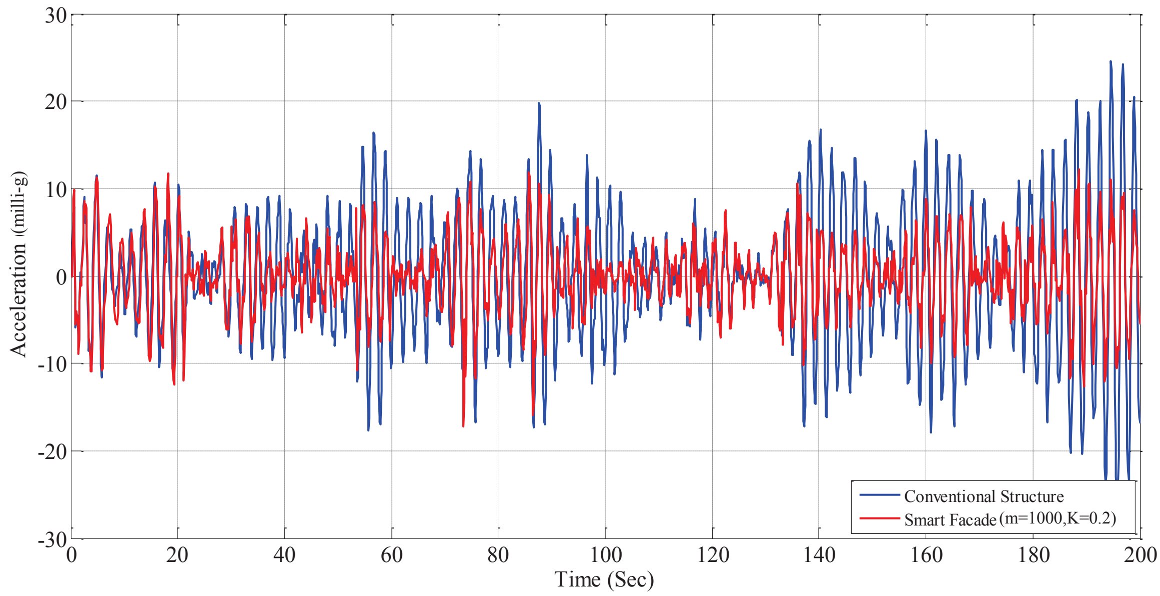


Figure 6.26 Performance of the conventional structure versus structure equipped with smart façade (m=1000, K=0.2kN/mm)

As stated earlier, increasing the weight of the panels is not necessarily the best way of having the best performance. As shown in Figure 6.26 although it is possible to get acceptable response, but it is not the best one as illustrated in Figure 6.27.

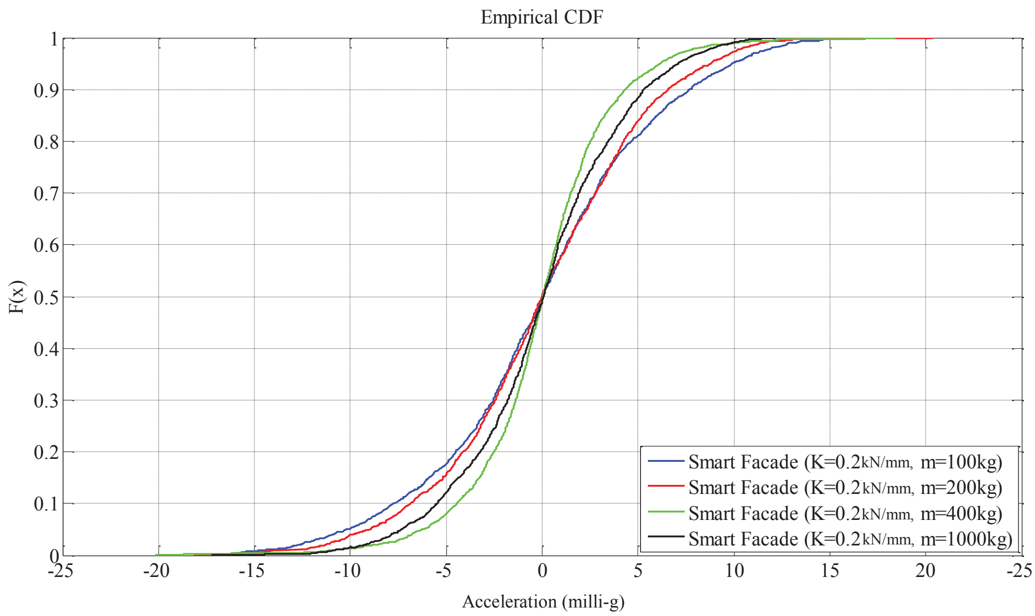


Figure 6.27 Cumulative density function of smart façade response, for 100kg, 200kg and 400kg and 1000kg per panel weight

Table 6.2 presents the average behaviour of each system which makes the comparison easier. The reduction trend is seen with increasing the panel weight up to 400kg panels. It is worth nothing that the smart façade concept is not just based on tuned mass damper theory, but the number of crossings and frequency of panels are also parameters which need to be considered.

Table 6.2 Standard deviation of the response of structures equipped with different facade dampers

	Conventional	K=0.2kN/mm m=100kg	K=0.2kN/mm m=200kg	K=0.2kN/mm m=400kg	K=0.2kN/mm m=1,000kg
Acceleration (milli-g)	7.697	5.823	5.258	3.702	4.246
Displacement (mm)	9.41	7.03	6.32	4.56	5.52

Midrise buildings can also be wind sensitive and controlling the top acceleration response may be needed to ensure occupant comfort. It can be concluded that a smart system could also have a significant effect on top displacement response of the structure. Moreover, the reduction trend of acceleration and displacement are quite similar but the system has better performance in reducing acceleration than displacement.

A midrange stiffness of 0.5kN/mm has been chosen to compare the performance of the structure in terms of different panel weights. The following, Figures and table present the effect of the higher stiffness and show the effect of the weight of the panels in this range. Figure 6.28 compares the performance of the conventional structure (blue line) with smart structure façade (red line). In this Figure each façade panel weight is 100kg and the soft stiffness is set to 0.5kN/mm.

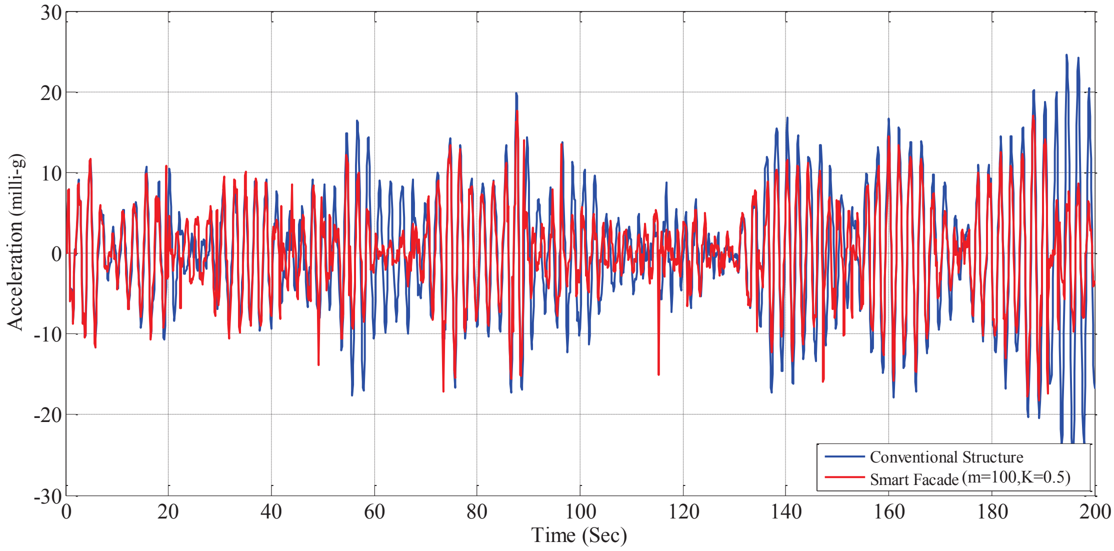


Figure 6.28 Performance of the conventional structure versus structure equipped with smart façade (m=100, K=0.5kN/mm)

By comparing the two responses in Figure 6.29 (façade with k=0.2kN/mm and k=0.5 kN/mm) it is interesting to note that by increasing the stiffness from 0.1 to 0.5kN/mm and maintaining the same mass in each panel as 100 kg, the structural response in both cases remain similar.

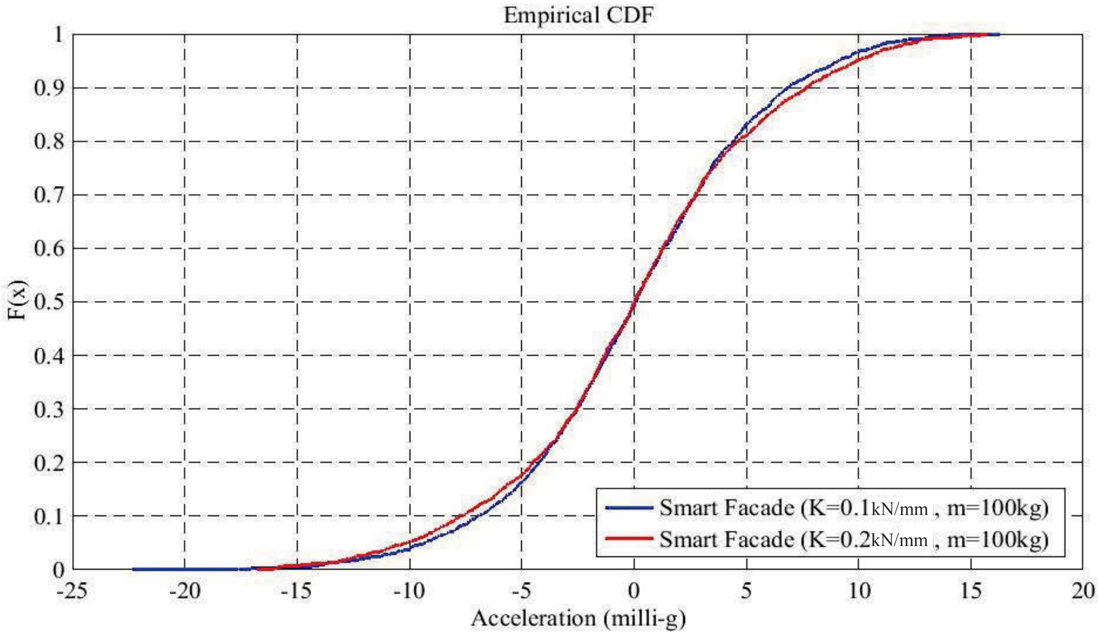


Figure 6.29 Cumulative density function of smart façade response, considering K=0.2 kN/mm and 0.5 kN/mm with 100 kg per panel weight

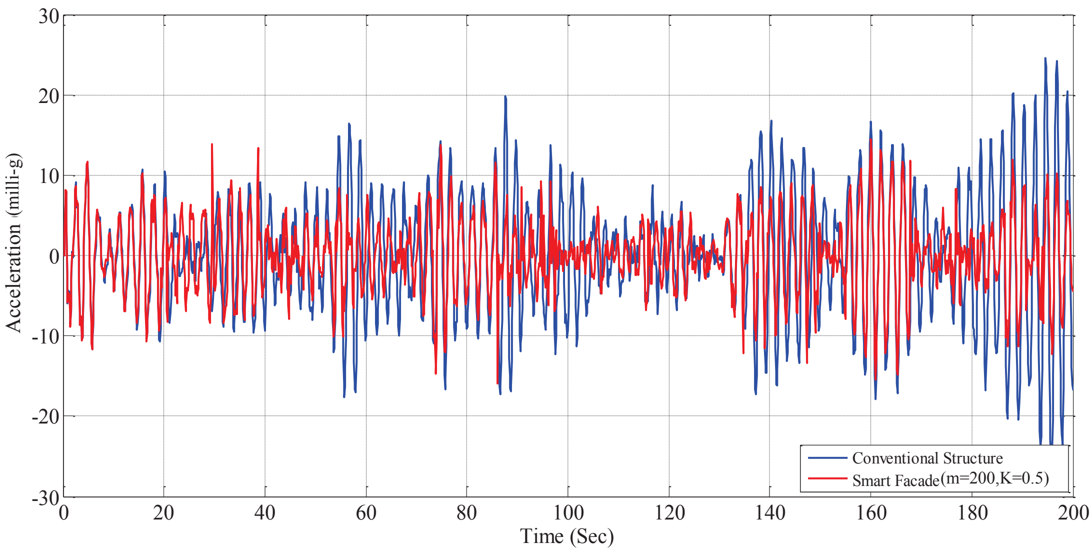


Figure 6.30 Performance of the conventional structure versus structure equipped with smart façade (m=200, K=0.5kN/mm)

It can be seen that by using 200 kg panels, top acceleration meets the comfort limits for residential buildings of close to 10 mg (Figure 6.30). Figure 6.31 shows that the combination of 0.5 kN/mm stiffness and 200 kg per panel performed better comparing to lower weight panels.

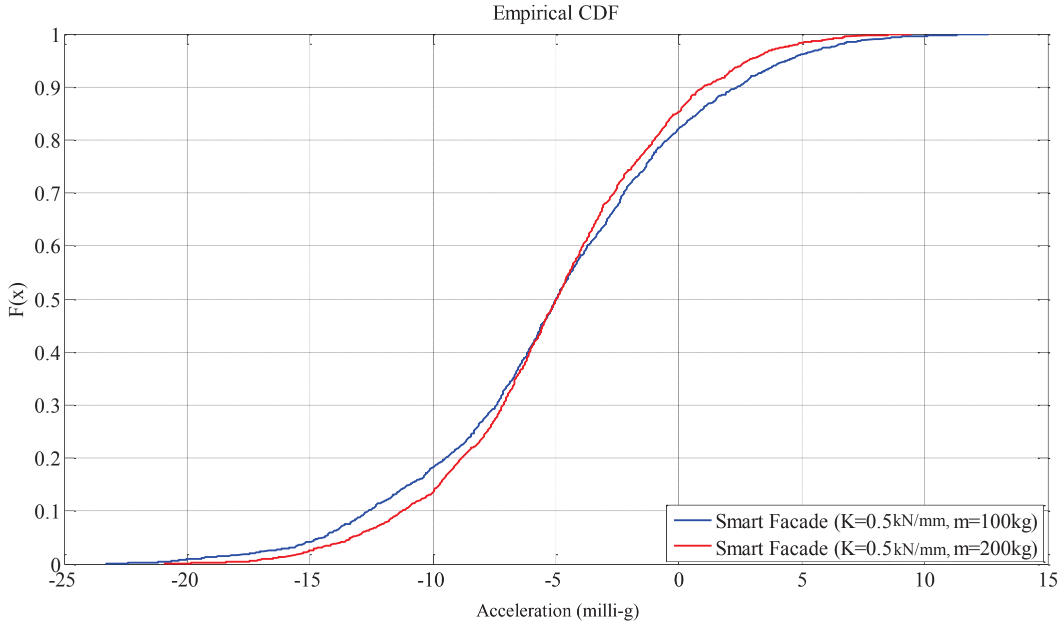


Figure 6.31 Cumulative density function of smart façade response, assuming 100kg, 200kg per panel weight

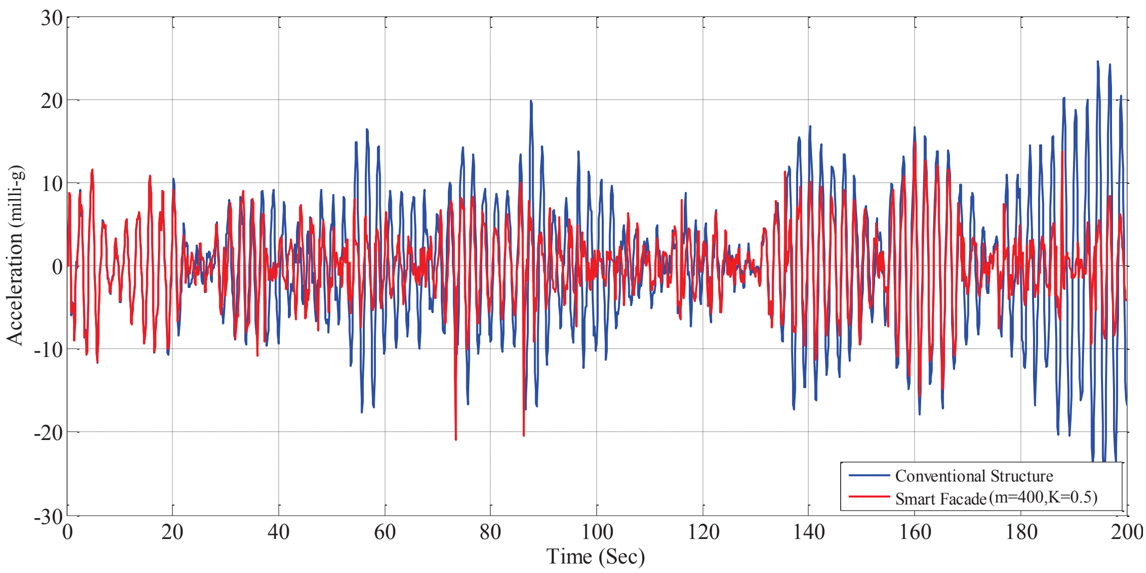


Figure 6.32 Performance of the conventional structure versus structure equipped with smart façade (m=400, K=0.5kN/mm)

Typical weight for façade panels were used to present the most realistic case. It is noticeable that from 40 second to 100 second of response in Figure 6.32, the structure with smart façade behaved more efficiently. Figure 6.33 indicates the difference of performance of systems in terms of weight of each panel. It can be seen that there is little difference between using 200 kg and 400 kg panels.

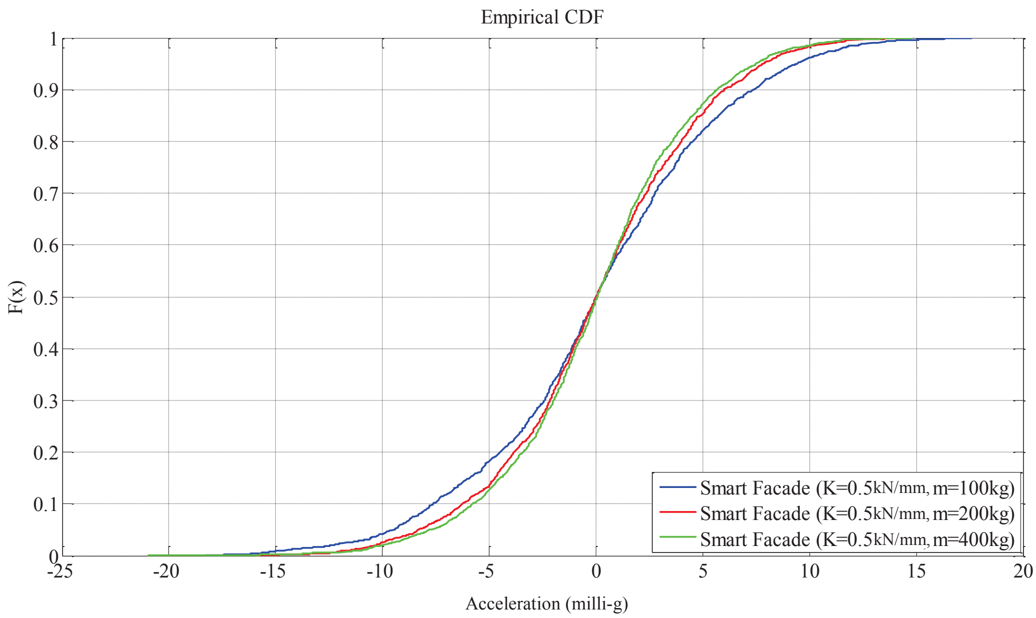


Figure 6.33 Cumulative density function of smart façade response, for 100kg, 200kg and 400kg per panel weight

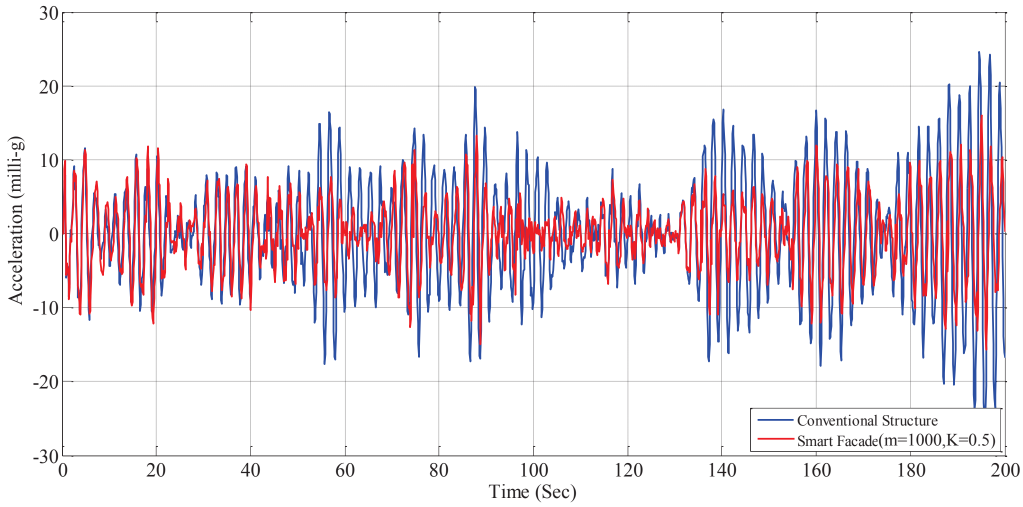


Figure 6.34 Performance of the conventional structure versus structure equipped with smart façade (m=1000, K=0.5kN/mm)

Up to now the heaviest glass façade panel used in construction is about 1000 kg per panel. A sensitivity analysis is undertaken in order to show the performance of smart dampers with 1000 kg panel, using 0.5 kN/mm soft stiffness. It is worth nothing that 1000 kg for each panel could not be effective in terms of top acceleration response reduction and also it could be shown that the combination using 1000 kg panel has similar effect compared to 200 kg panel. It can be concluded, therefore, that stiffness of 0.5kN/mm is not an effective combination (Figure 6.34 and 6.35).

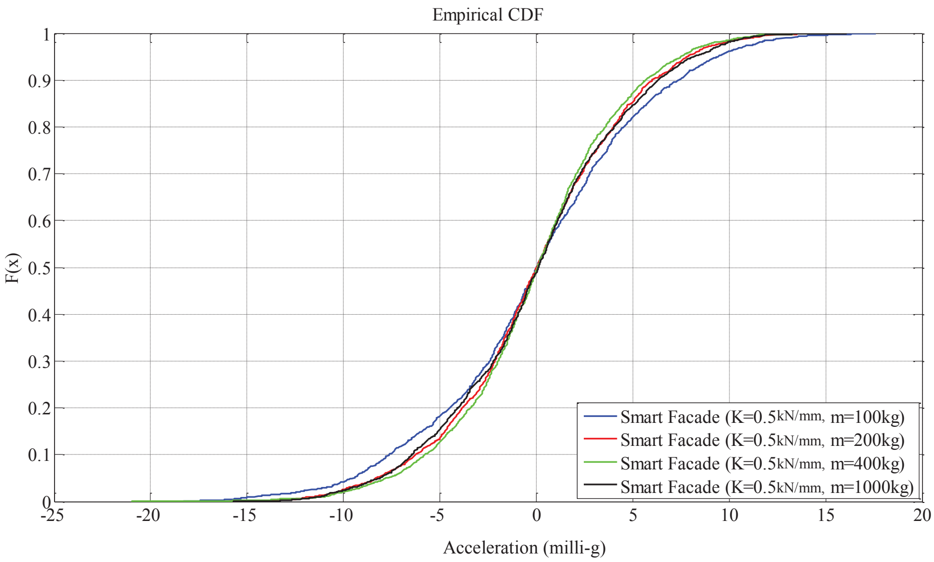


Figure 6.35 Cumulative density function of smart façade response, assuming 100kg, 200kg,400kg and 1000kg per panel weight

Table 6.3 Standard deviation of the response of structure equipped with different facade dampers

	Conventional	K=0.5kN/mm m=100kg	K=0.5kN/mm m=200kg	K=0.5kN/mm m=400kg	K=0.5kN/mm m=1000kg
Acceleration (milli-g)	7.697	6.257	7.029	7.681	7.054
Displacement (mm)	9.41	7.54	8.52	9.38	8.93

To summarize and assess the response of structure, standard deviation is used. All four combinations are presented in terms of acceleration and displacement in comparison with conventional structures. It can be shown that the beneficial effects are negligible using 0.5 kN/mm as stiffness. It is worth nothing that using different panel weights is not so effective for top acceleration reduction.

As stated earlier, the number of crossing a particular threshold is one of the important criteria that leads to decrease of the response of structures. To meet the required crossing frequency, it is essential to have the right combination of stiffness and panel mass. Sensitivity analyses is performed using low medium, and high stiffness as soft stiffness. It is quite important to see how sensitive is the system in terms of crossing frequency and if so what is the optimum value for mass and stiffness of the façade damper system. The following results are presented using relatively high stiffness.

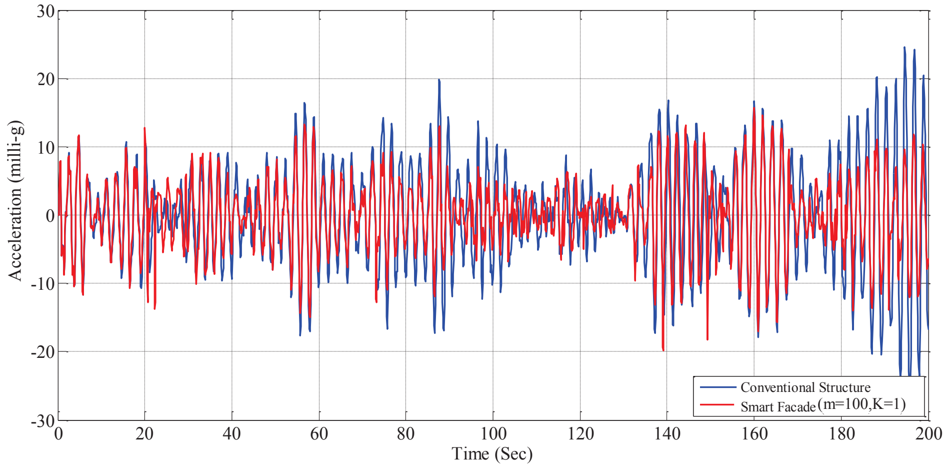


Figure 6.36 Performance of the conventional structure versus structure equipped with smart façade (m=100, K=1kN/mm)

It is clear that increasing the stiffness in the second phase (soft stiffness) leads to negligible benefits compared to conventional structures. Figure 6.36 shows the top acceleration response when the façade panels are 100kg and the soft stiffness is equal to 1kN/mm. Figure 6.37 indicates the difference of performance of systems in terms of stiffness of each panel. It can be seen that there is little difference between them.

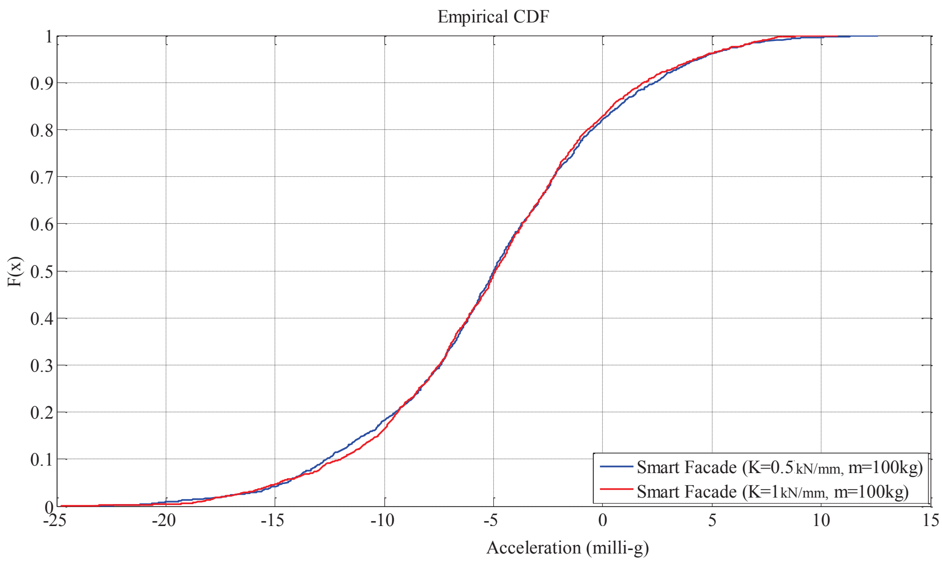


Figure 6.37 Cumulative density function of smart façade response, considering K=0.5kN/mm and 1kN/mm with 100kg per panel weight

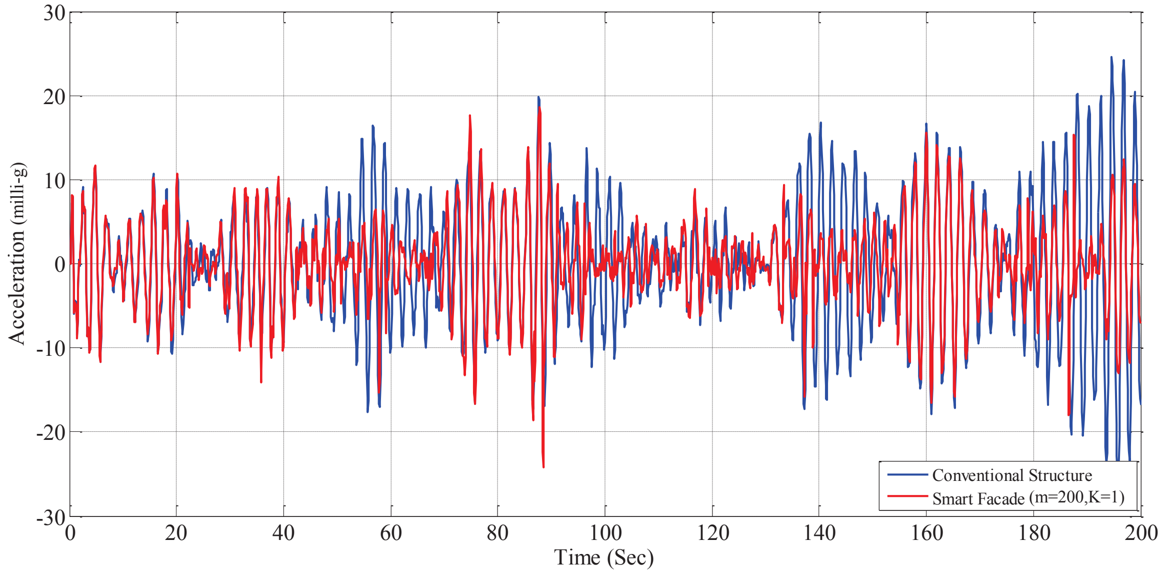


Figure 6.38 Performance of the conventional structure versus structure equipped with smart façade (m=200, K=1kN/mm)

By increasing the weight of panels it is expected that the performance of the system drop, although this drop is small compared to the previous case with 0.5kN/mm (Figure 6.38). Figure 6.39 shows the comparison between the system with 100kg panel and 200kg panel. It can be concluded that increasing the weight of the panels by 100kg dose not lead a better performance of the system in terms of response of structures.

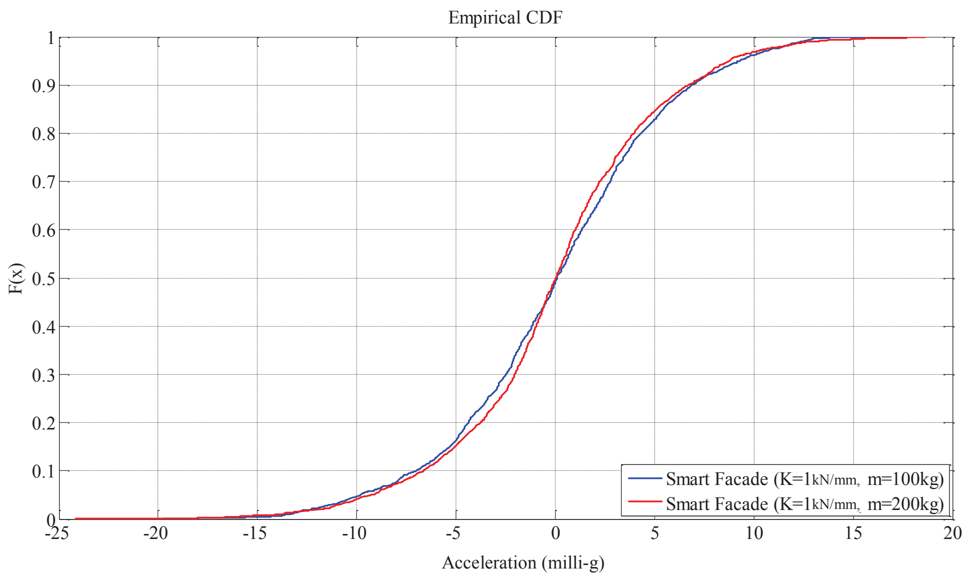


Figure 6.39 Cumulative density function of smart façade response, assuming 100kg, 200kg per panel weight

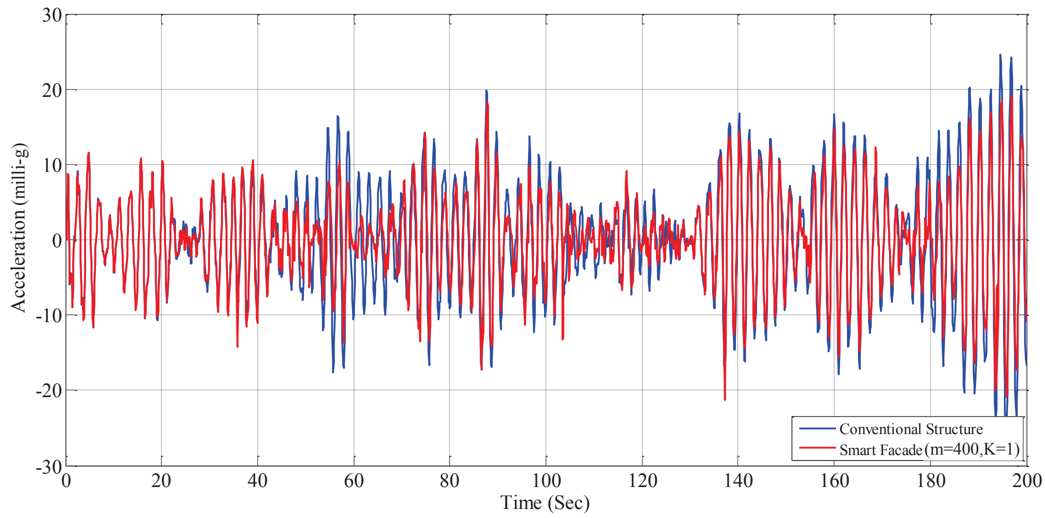


Figure 6.40 Performance of the conventional structure versus structure equipped with smart façade (m=400, K=1kN/mm)

Choosing 400kg as a weight of panels lead to push the smart façade system response close to the conventional facade response (Figure 6.40). In order to compare the response of 100kg, 200kg and 400kg panel Figure 6.41 is presented. It is notable that the best effect is occurring when the facade is 200kg weight and also equipped with 1kN/mm stiffness value in second slope.

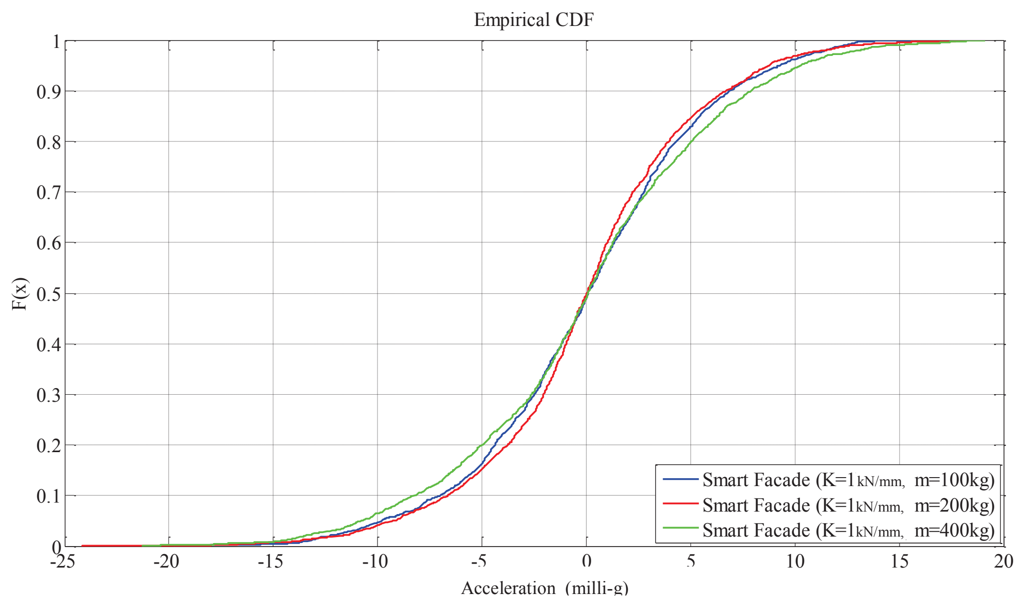


Figure 6.41 Cumulative density function of smart façade response, assuming 100kg, 200kg and 400kg per panel weight

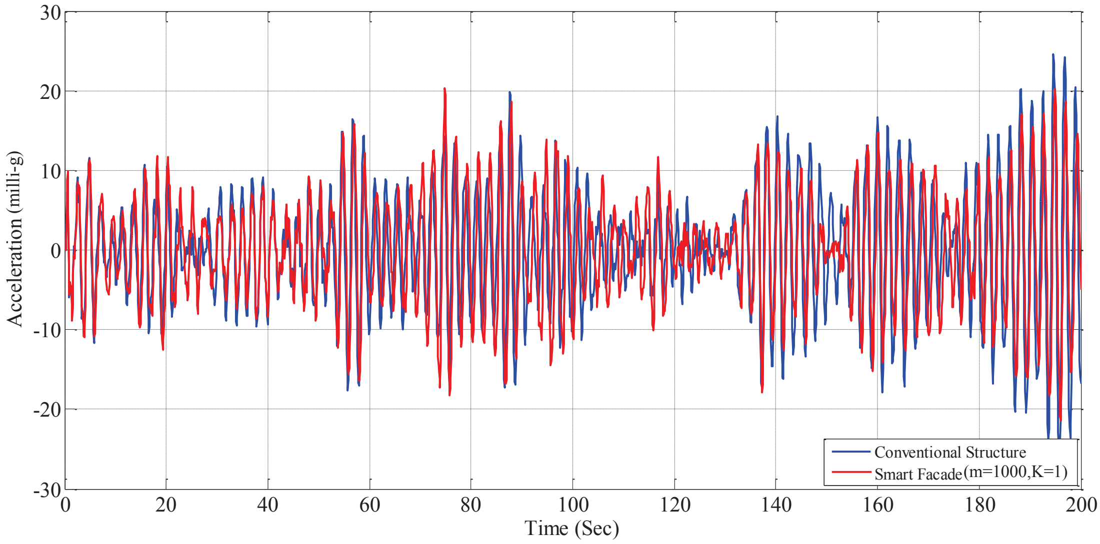


Figure 6.42 Performance of the conventional structure versus structure equipped with smart façade (m=1000, K=1kN/mm)

From the previous results, it is expected that increasing the weight of the panels to 1000kg lead the response to omit the beneficial effect of the system. The goal of using smart material and changing the conventional concept is to reduce and come up with a better response in terms of acceleration and displacement of the structure, however, it could be shown that this adjustment in some part leads to excessive acceleration which is not acceptable (Figure 42 and 43).

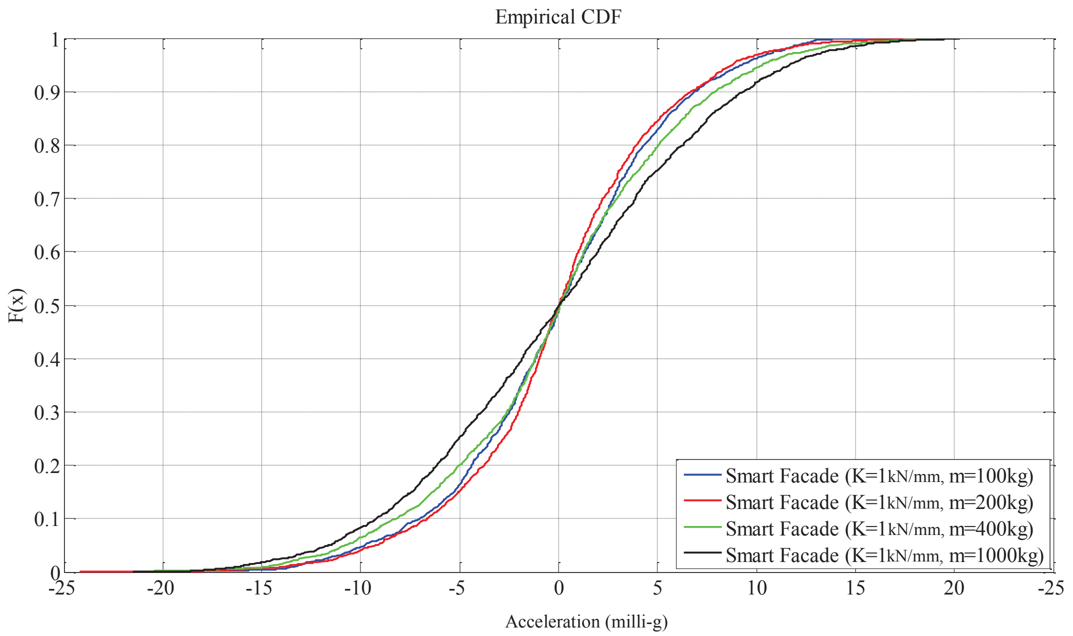


Figure 6.43 Cumulative density function of smart façade response, assuming 100kg, 200kg, 400kg and 1000kg per panel weight

To provide a relative comparison between all the damper adjustment standard deviation of each response is presented in table 6.4. It can be shown that the beneficial trend goes down by increasing the weight and also this logic can be extended to displacement response as well.

Table 6.4 Standard deviation of the response of structure, which equipped with different facade damper

	Conventional	K=1kN/mm m=100kg	K=1kN/mm m=200kg	K=1kN/mm m=400kg	K=1kN/mm m=1000kg
Acceleration (milli-g)	7.697	5.492	5.339	6.248	7.113
Displacement (mm)	9.41	6.61	6.43	7.68	8.98

Due to large differences in stiffness ratio between the first slope and second slope of the smart damper behaviour, it is logical to do some sensitivity analysis which lead to have the optimum value. Although the minimum effects system should reach is reducing the 20% of the top response of structures.

Increasing the stiffness value to 2kN/mm in second slope is enough to come up with poor design damper façade as it has marginal impact on the response of structures. The following Figures are meant to present the behavioural comparison of structures equipped with different weight of the panel using 2kN/mm stiffness value. Figure 6.44 compares the performance of the conventional structure (blue line) with smart structure façade (red line). In this Figure each façade panel weight is 100kg and the stiffness value is 2kN/mm for the soft part.

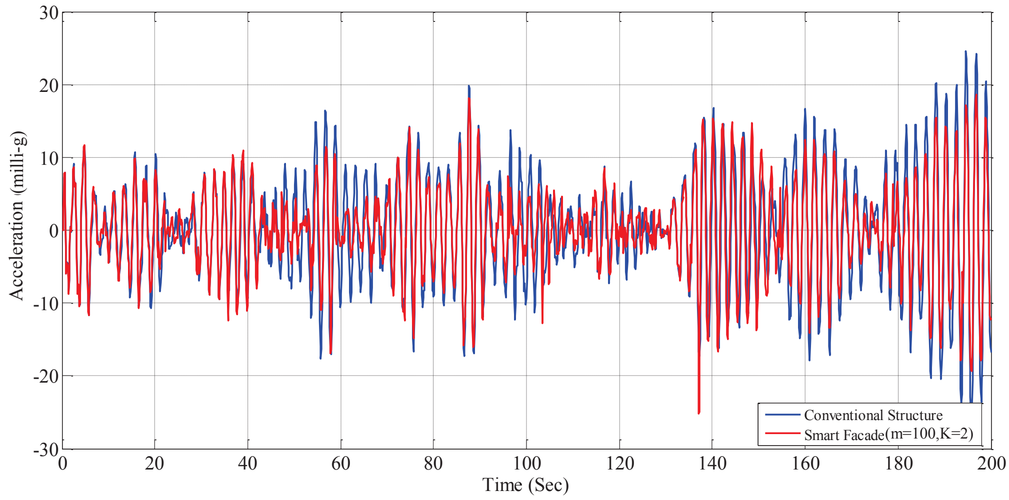


Figure 6.44 Performance of the conventional structure versus structure equipped with smart façade (m=100, K=2kN/mm)

It can be also seen that in Figure 6.45, the combination of 100kg weight for each façade panel and 2kN/mm for the second slope ratio leads to small difference in response of structures compared to the case with half slope ratio. The only change which occurs in the response is between 5mg to 15mg of the behaviour. It is worth to mention that the difference between 1kN/mm to 2kN/mm damper stiffness is negligible.

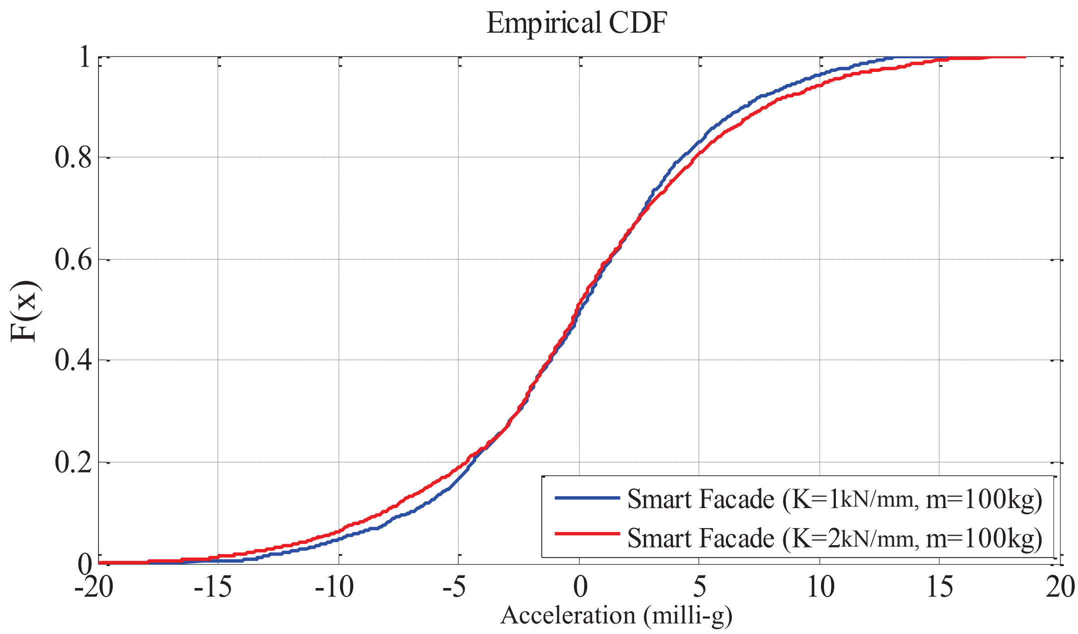


Figure 6.45 Cumulative density function of smart façade response, considering K=1kN/mm and 2kN/mm with 100kg per panel weight

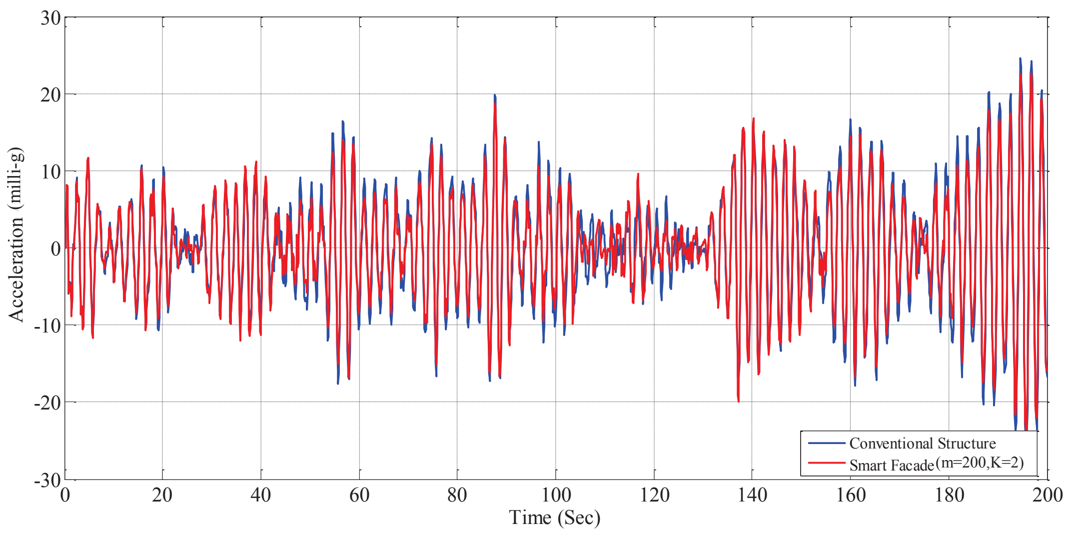


Figure 6.46 Performance of the Conventional Structure versus Structure Equipped with Smart Façade (m=200, K=2kN/mm)

Keeping the stiffness of the dampers in second slope as 2kN/mm and increasing the façade panel weight to 200kg leads the panels to vibrate out of phase. In fact, this is the cause to a small difference between the conventional structure façade system and the presented combination of smart system façade (Figure 6.46). As it is illustrated in Figure 6.47 the difference which is the result of weight change is slightly more noticeable than the change caused by slope difference.

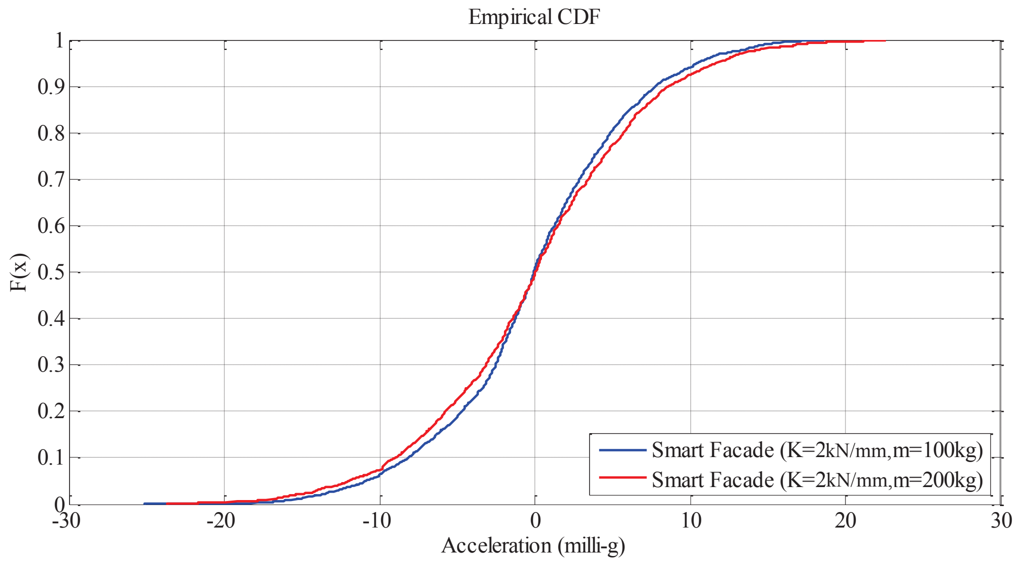


Figure 6.47 Cumulative density function of smart façade response, assuming 100kg, 200kg per panel weight

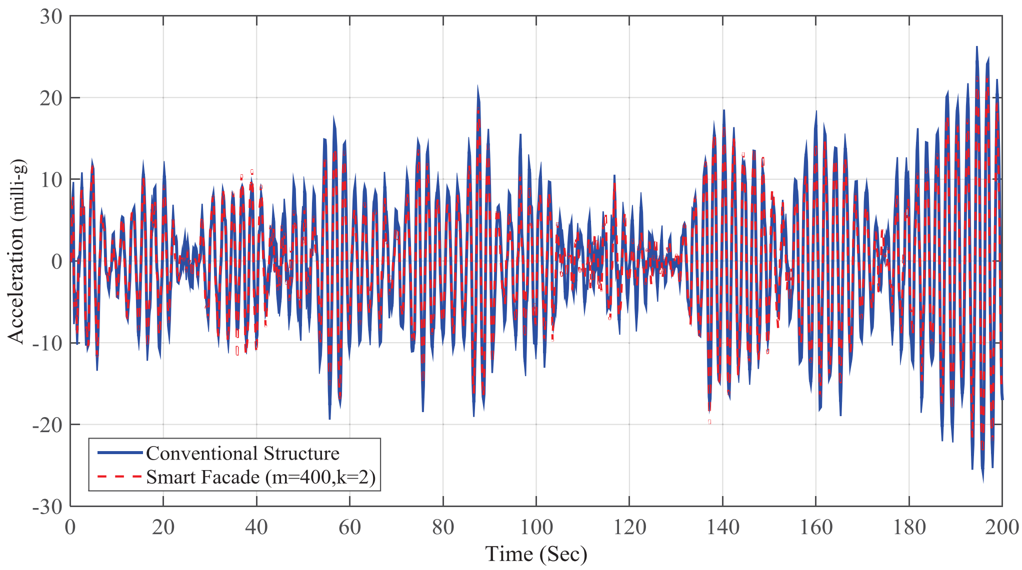


Figure 6.48 Performance of the conventional structure versus structure equipped with smart façade (m=400, K=2kN/mm)

While there is a slight difference between the conventional façade system and the smart façade system with 200kg panel, when it comes to 400kg panel, which is typically used in industry there is absolutely no reduction in the response of the structure. It appears to be reasonable to point out that the weight difference up to 400kg per panel is not considerable economic wise (Figure 6.48 and 6.49).

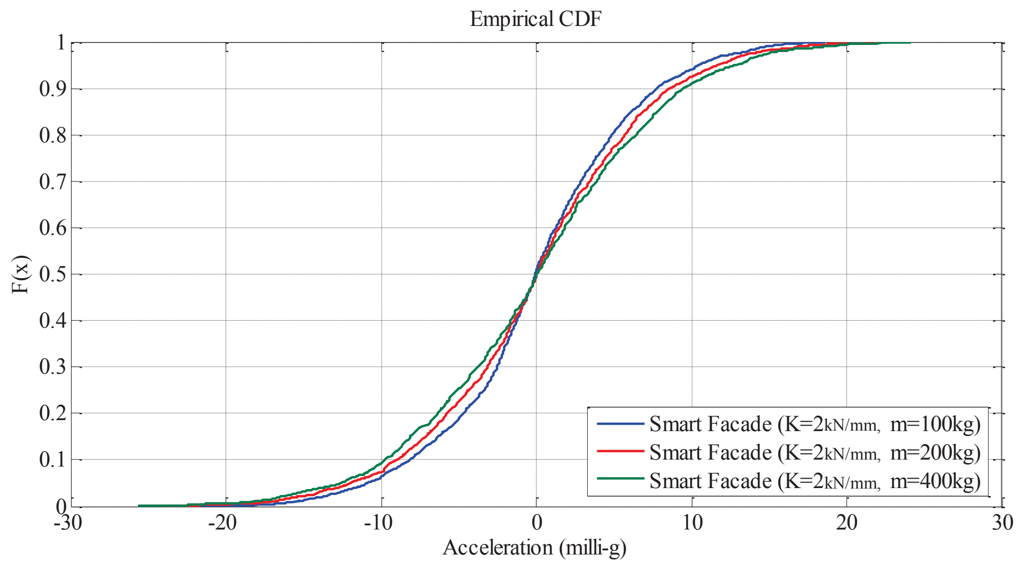


Figure 6.49 Cumulative density function of smart façade response, assuming 100kg, 200kg and 400kg per panel weight

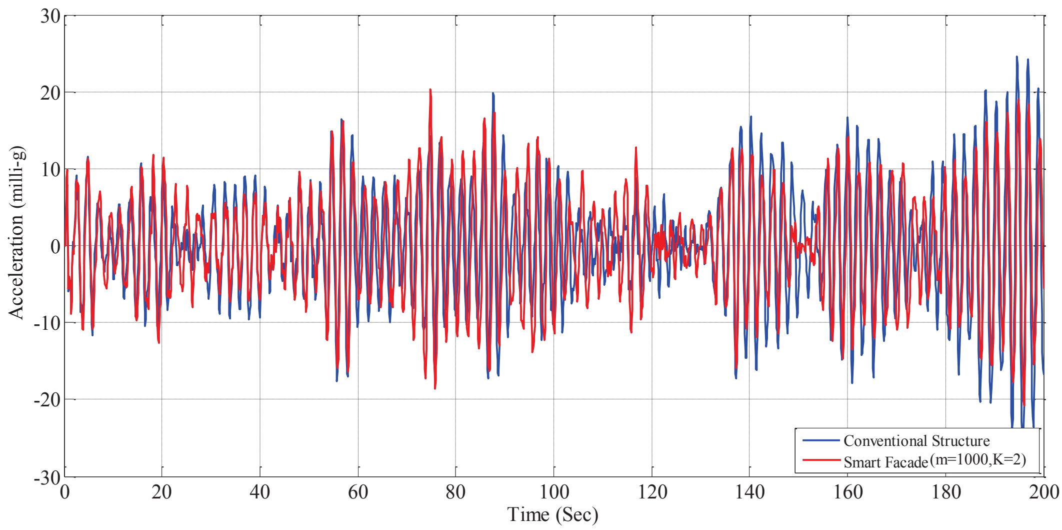


Figure 6.50 Performance of the conventional structure versus structure equipped with smart façade (m=1000, K=2kN/mm)

Upon increasing the weight to 1000kg, the smart system begins to make changes in the performance of the structure which at times reduces its acceleration response and sometimes it remains unchanged (Figure 6.50). It is evident that the cumulative density function is in accordance with the above discussion (Figure 6.51).

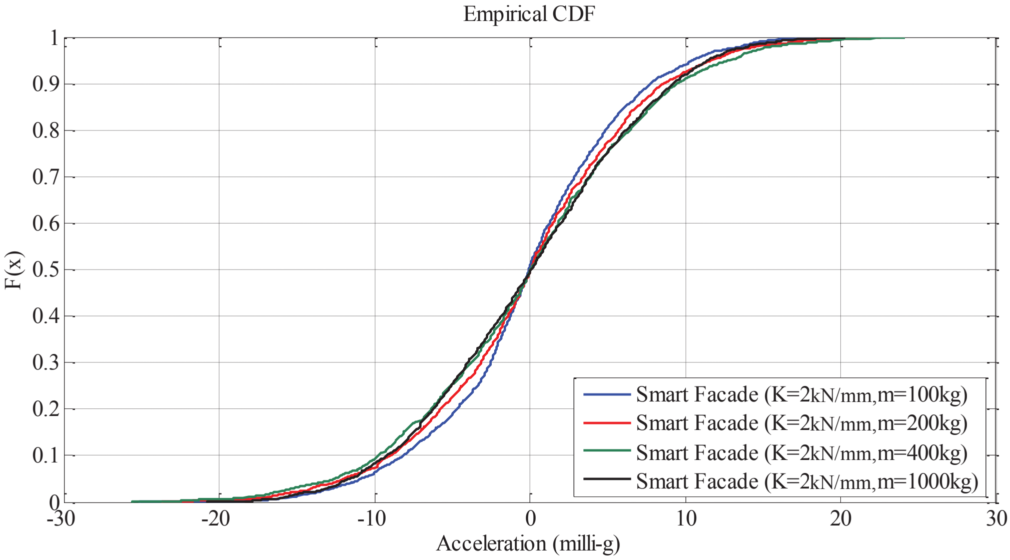


Figure 6.51 Cumulative density function of smart façade response, assuming 100kg, 200kg, 400kg and 1000kg per panel weight

In line with the data presented in table 6.5, reduction trend is matched with increasing the panel weight, except for 400kg panels. It is not only the tuned mass damper theory, but also other parameters as in the number of crossing and the frequency of panels, which are to be considered.

Table 6.5 Standard deviation of the response of structure which equipped with a different facade damper

	Conventional	K=2kN/mm m=100kg	K=2kN/mm m=200kg	K=2kN/mm m=400kg	K=2kN/mm m=1000kg
Acceleration (milli-g)	7.697	6.623	7.124	7.325	7.425
Displacement (mm)	9.41	7.64	8.52	8.88	9.13

6.5 Sensitivity Analyses Based on the Length of Second Slope (soft stiffness)

Movable façade is an attractive concept and could have significant effects on the response of structures. The recent works have identified the excessive movement of the façade skin to be an effective system. The supporting analysis identified a practical challenge, namely requiring the panels to move several meters to be effective. In this research, multi linear behaviour damper is used instead to control the large displacement of the panels and also display a similar reduction in the response of the main structure. The soft stiffness part in the façade damper behaviour system leads the system to behave like a multi tuned mass damper system. Previous researchers have concluded that the frequency of the movable facade should be around half of the main structure, frequency to be effective which requires a relatively low stiffness connections between façade skin and the main structure. Therefore, a sensitivity analysis is presented here to show the optimum length for the second slope needed in order to have an effective system.

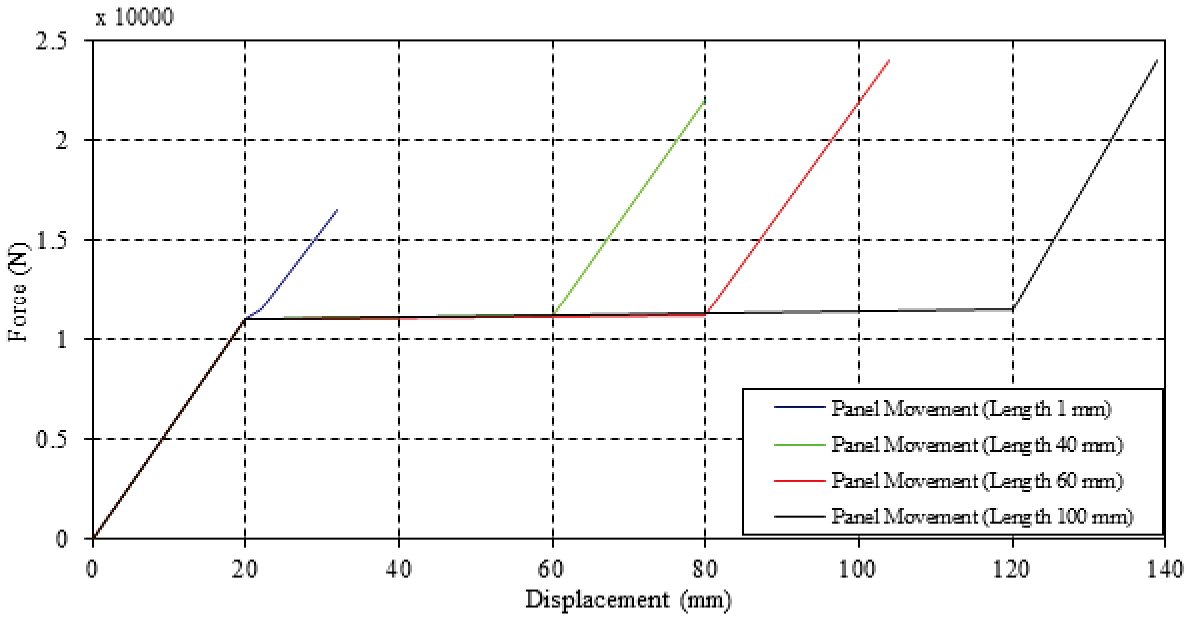


Figure 6.52 Panel movement, variable in length

Figure 6.52 shows four different cases to be studied in this section. The curves are meant to provide a relative comparison between the case studies which cover the range of second slope length from 1 mm to 100mm.

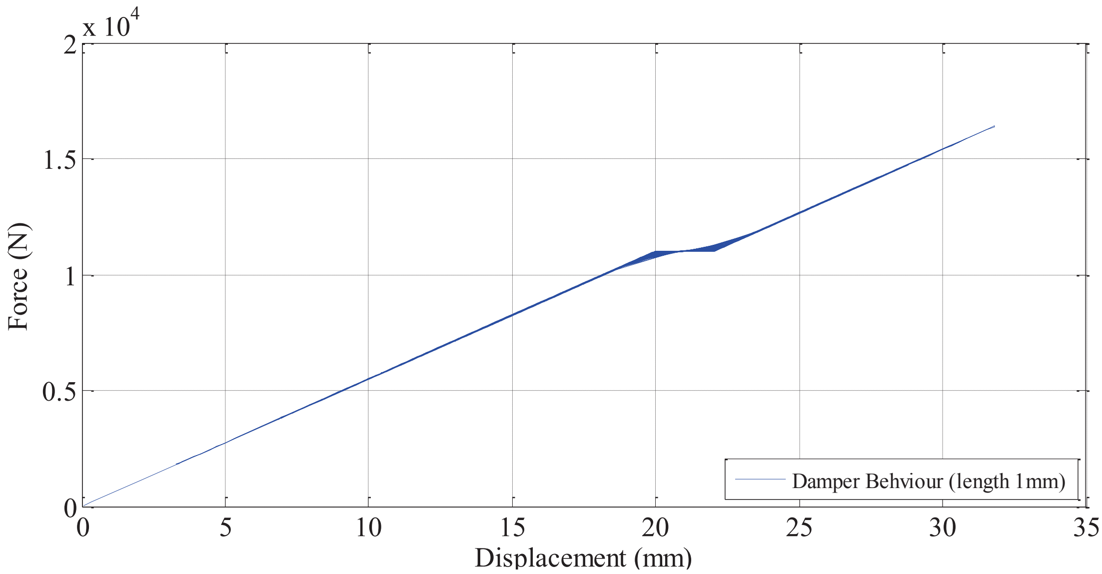


Figure 6.53 Damper behaviour when the second slope length is 1mm

As shown in Figure 6.53, damper panel is assumed to have 1mm of soft deformation in the second slope. It can be seen from Figure 6.54 that small distortion in linear behaviour does have a slight effect on the response of structures.

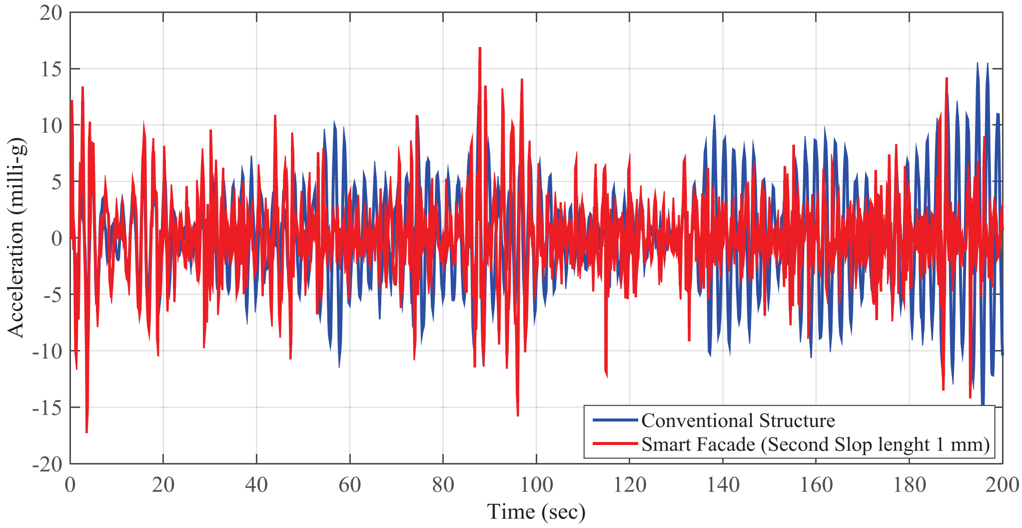


Figure 6.54 Time history response of conventional vs smart façade when the second slope has 1 mm length

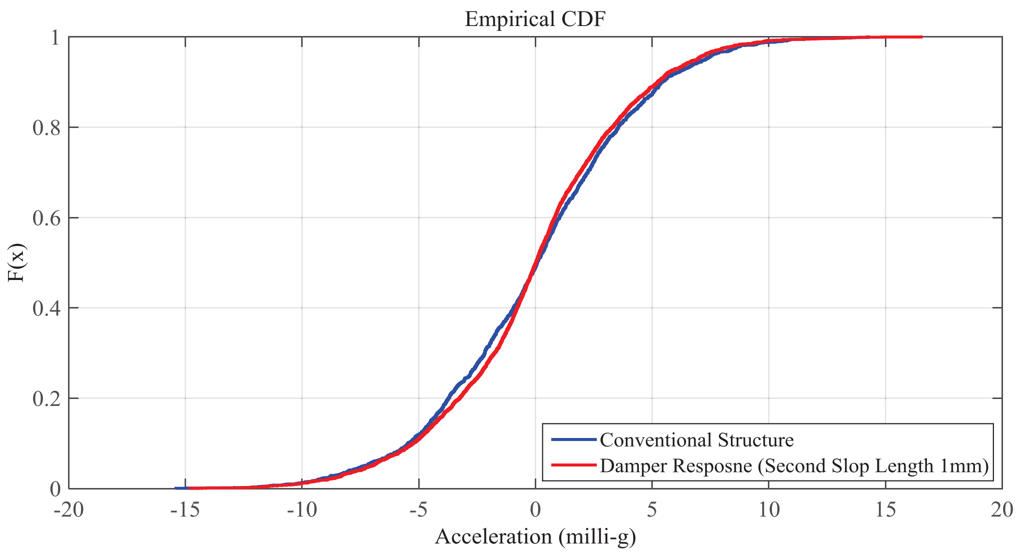


Figure 6.55 Comparing the cumulative density function of top acceleration response with second slope length of 1 mm

Figure 6.55 presents the comparison of structure by using the cumulative density function when the second slope has 1 mm to move. It is worth nothing that the standard deviation of acceleration in conventional structure is equal to 43.64 milli-g which almost similar to 41.33 milli-g for structure with damper façade. It can be concluded, in order to have a better system performance, the panels should be able to move in a larger range (Figure 6.56).

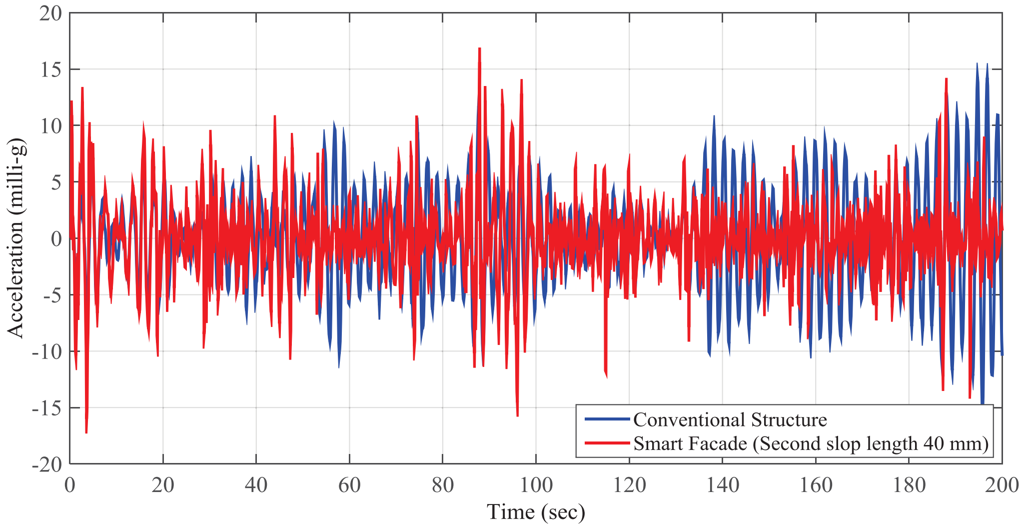


Figure 6.56 Time history response of conventional vs smart façade with second slope length of 40 mm

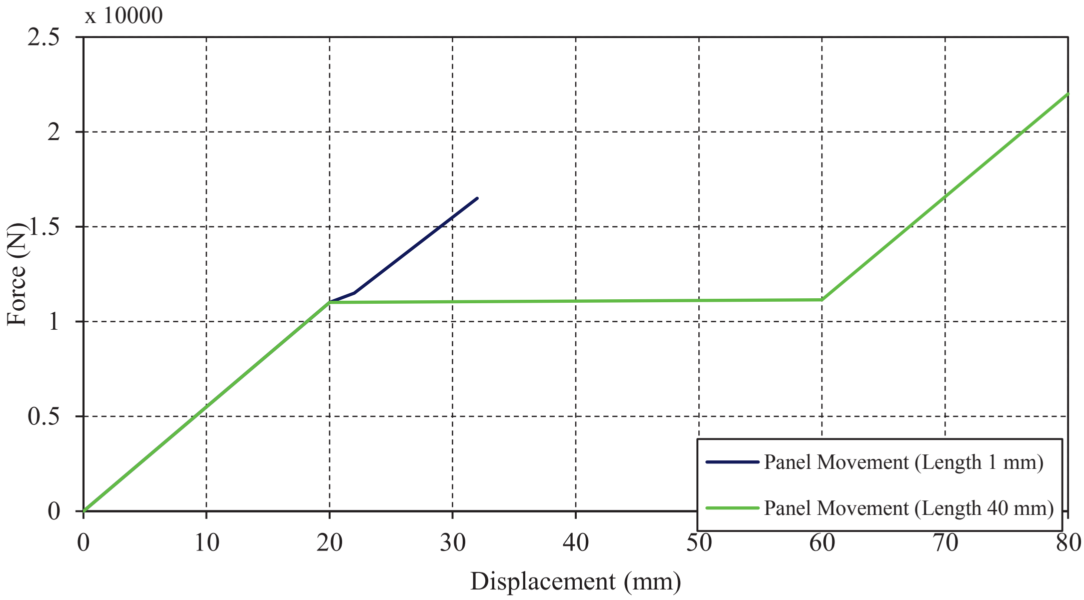


Figure 6.57 Comparing panel movement when the second slope length is 1mm versus 40mm

It can be demonstrated that by increasing the gap-length between two stiff paths to 40 mm the system performance starts to show some beneficial effect (Figure 6.57). The behaviour of dampers shown in Figure 6.58 is compared when the second slope changes from 1mm to 40mm length. It can also be seen from Figure 6.59 that obtaining 20% reduction in acceleration response is achievable.

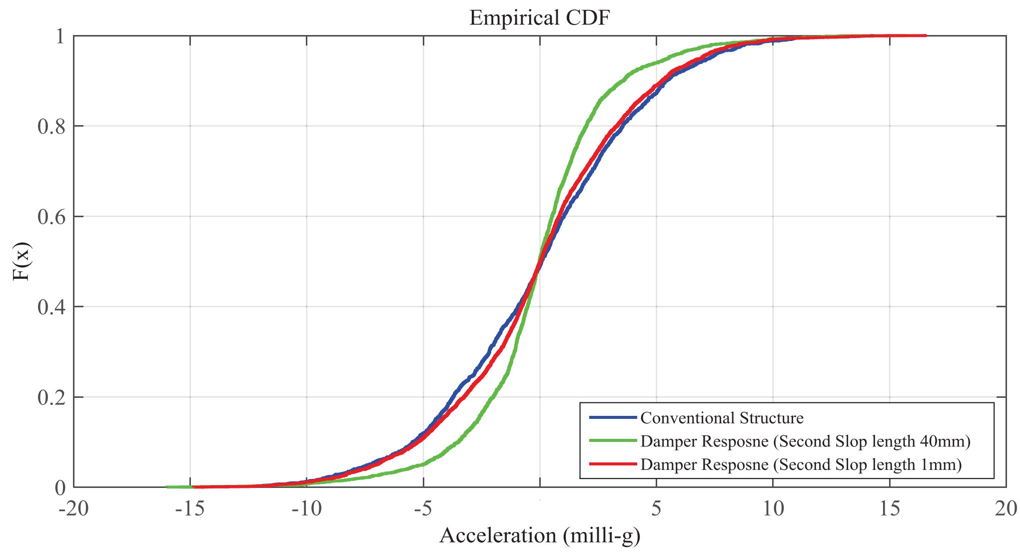


Figure 6.58 Comparing the cumulative density function of top acceleration response when the second slope of 40 mm in length

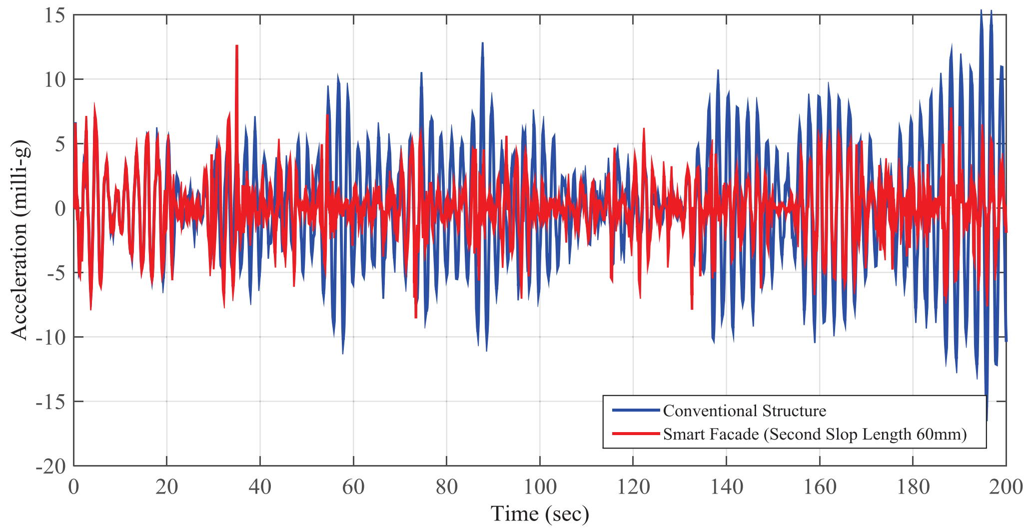


Figure 6.59 Time history response of conventional vs smart façade when the second slop has 60 mm length

The longer gap-length between two stiff paths, allows the panel to get closer to the right frequency. The aforementioned matter is crucial in reducing the primary structure response. Comparison of time history response of conventional versus smart façade structure is shown in Figure 6.60. In terms of acceleration response, 30% reduction is achievable by increasing the length of soft path to 60 mm.

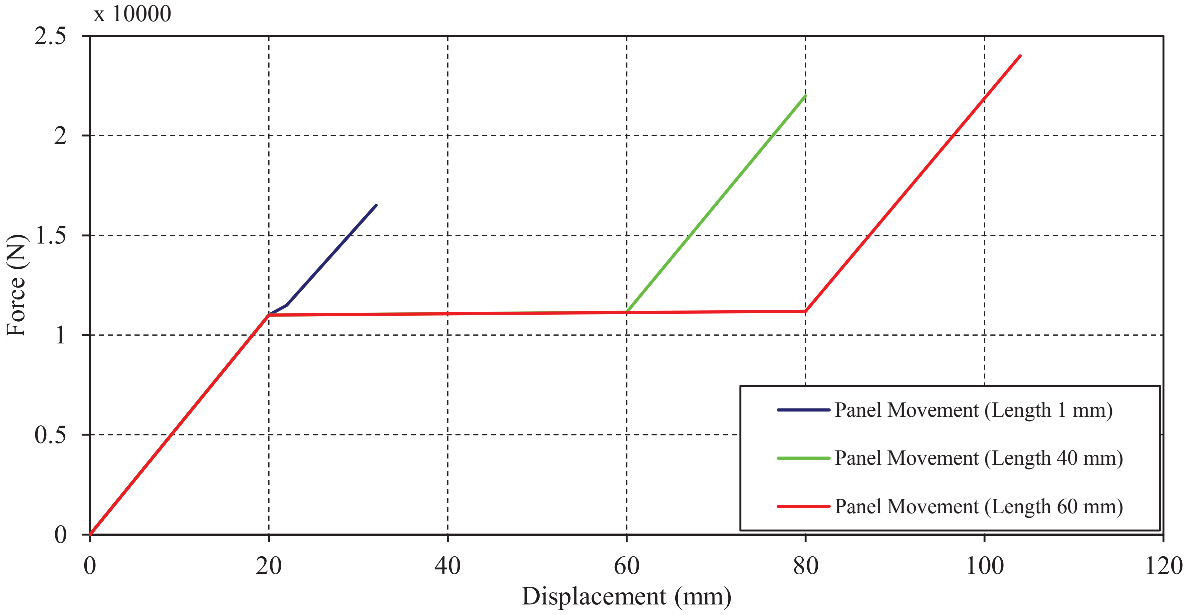


Figure 6.60 Comparing panel movement when the second slope length is 1mm versus 60mm

When smart damper reach certain frequency, acceleration response reduction appear. To get the right frequency, 100 mm of soft path is needed. Time history response of conventional versus smart façade structure is shown in Figure 6.61. This figure represents the maximum reduction that can be achieved by using moveable façade panels. reduction that can be achieved by using moveable façade panels.

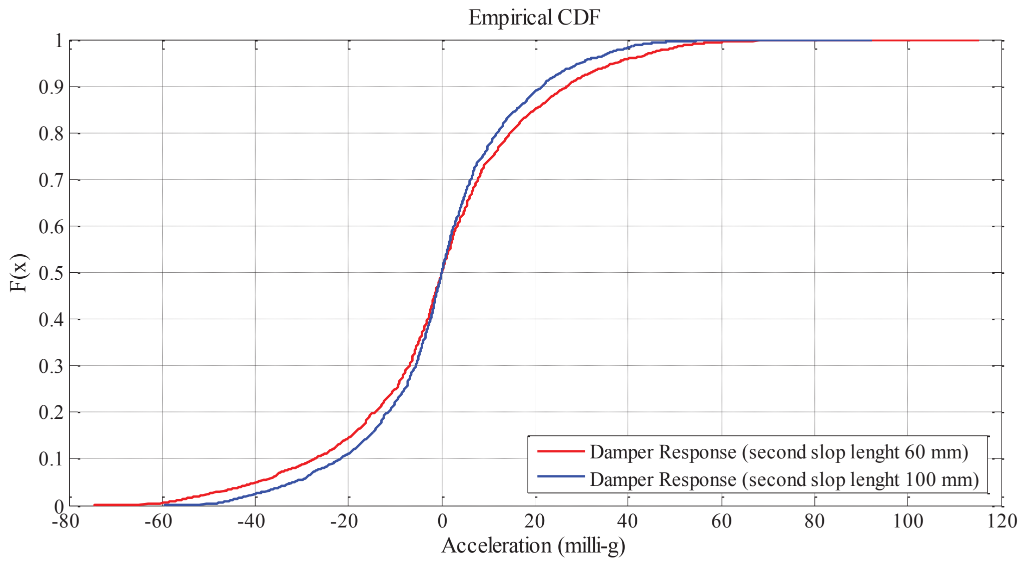


Figure 6.61 Comparing the cumulative density function of top acceleration response when the second slope of 40 mm in length

6.6 Figure Gain of System

A damper system designed based on yearly wind, which makes some limitations in terms of beneficial effects. This system works in a range of mean wind speed and it can be demonstrated based on sensitivity analysis which is done through different mean wind speed. As shown in Figure 6.62 series of analysis were done to determine the system gain, which is possible to have partly 50% reduced. It is worth noting that while the performance could not cover the whole mean speed, but it could be really effective in a certain range.

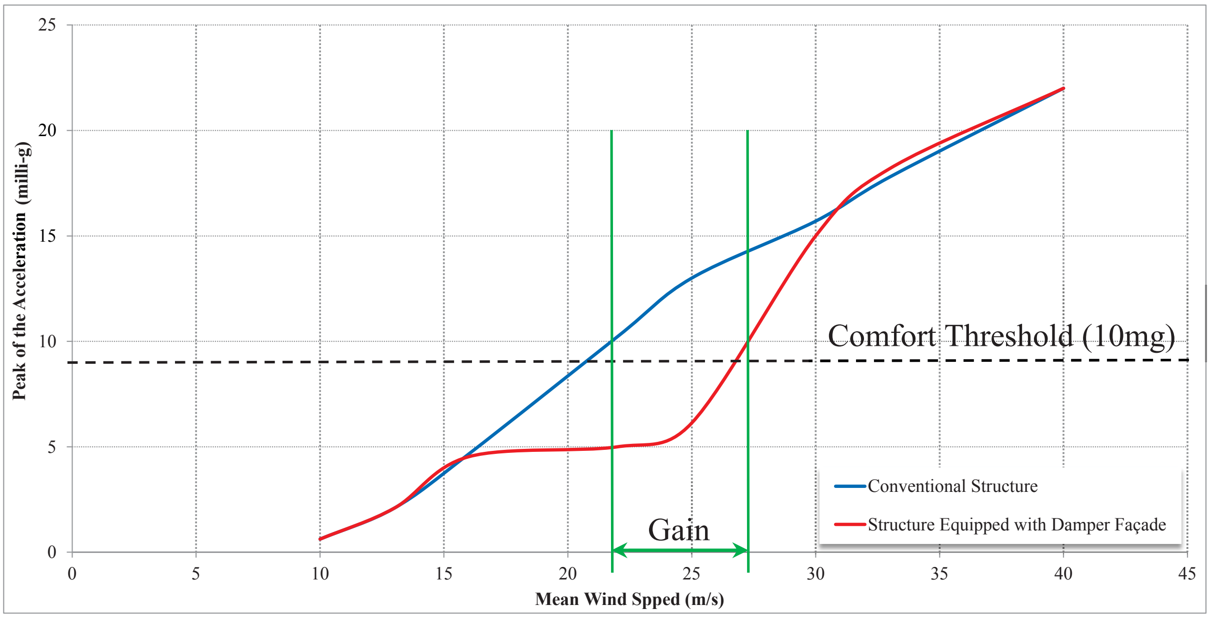


Figure 6.62 The damper façade beneficial effect trend

6.7 Conclusion

To measure effectiveness of façade dampers, specific analyses have been carried out. Linear and multi linear behaviour of façade damper with respect to displacement history of façade panels are considered. To summarize, low stiff path should be designed in a way that façade panels reach the right frequency to dissipate the input energy.

In this chapter, the effect of mass and stiffness ratio was also covered. Coupling different façade panel weight with various stiff path leads to sensitivity analyses. It should be noted that, this concept is based on panel frequency which makes this system sensitive to stiffness of the dampers and the weight of panels. In conclusion, 400 kg as a panel weight and 0.2 N/mm showed the optimised behaviour.

Further analyses have been presented to show the optimum low stiff path length needed to ensure the effectiveness of the system. Sensitivity analyses include different length; from 1 mm to 100 mm. This length is needed to let the panels reach the right frequency. The aforementioned phenomena occur when the soft path length is 100 mm.

In the last part of this chapter, the response trend of a structure subjected to the range of wind load is presented in order to describe the gain value of the system.

Chapter 7

7 Financial

7.1 Introduction

The effect of high winds on high rise buildings is well recognised by the engineering communities. High-rise buildings may be susceptible to excessive deflections and perceptible accelerations and such problems usually require external damper systems as a remedy under wind loads. The loss of valuable and prime space coupled with the initial cost of installing large size damper systems has been accepted by building owners with reluctance and any viable alternative system to dissipate wind excited vibrations will be welcomed by the building owners. It is proposed to use a movable façade skin attached to passive devices which are in turn attached to a fixed frame within this investigation. The energy imparted by wind forces can be dissipated by the absorbing façade system with major economic benefits.

The research work has been concentrated on different types of flexible/energy absorbing façade systems and their behaviour under wind loading. A substantial number of numerical, analytical and experimental analyses related to the simulated wind performance of facade systems have been performed. The "smart" façade system involved is an energy absorbing one incorporating specially designed passive dampers. The ultimate goal of the research project is the design of a new energy absorbing façade system, fully tested against wind loads.

The study presented in this chapter is a summary of the research Work Package "Proof of concept under wind loading", where the wind loaded building with flexible/energy absorbing façade systems is compared with typical strengthened one in terms of economic benefits.

This financial study is not describing all concepts that were developed, simulated and analysed during the proofing period, but it emphasizes uniquely the concept that demonstrated feasibility and potentiality for business application. The numerical simulation results are presented and compared with the building with fixed facade, to highlight the benefit and improvement of the building having flexible/energy absorbing façade.

It should be clarified that the financial proof of the concept is done on simplified numerical models: the presented result accuracy should be taken with reserve - they represents the order of magnitude and they are useful to verify the concepts feasibility and to quantitatively compare different concepts.

This chapter includes five topics: the first part outlines a description of the major factors influencing the cost of a building. Obviously the location of the building is one of the most important ones and for this reason a further section shows the selected markets for the following comparisons. Then the list of the investigated cost breakdowns and their dependence on market region and other parameters is presented. Two test cases respectively focused on mid-rise and high-rise building examples have been defined and a methodology for the assessment of the façade damper benefit has been identified for better understanding of the reader. Finally, the results of the applied methodology to the two test cases are summarized in the last section.

7.2 Additional cost of the movable facade to building structure

7.2.1 Introduction

By introducing this state-of-the-art system, two additional expenses namely as design and maintenance of the proposed system are expected to add to overall cost of the building structure. As the system would be new to the façade industry and not many engineers and technical familiar with the system then, initial training and workshops need to be conducted to train them for dealing with the new system. It should be mentioned that, along with aesthetics, a façade must be designed to separate the interior of a building from the aggressive exterior environment and it must withstand the imposed mechanical and environmental loads. The service life of the proposed system starts when the construction work is finished. At this stage, bracket facades, especially the ones that are used in double skin façade systems, may deteriorate during their service life due to the effect of various aggressive environment mechanisms along with applied repetitive environmental loads. So, the additional costs of replacing the conventional façade system with the proposed system will be discussed in section 7.3.2 and 7.3.3.

7.2.2 Design or re-design procedure

Depending on building age and submitted request by the building owner, two possible scenarios are expected as out find below:

In the first scenario, dynamic analysis of an existing structure shows that the structure is under danger from upcoming winds with a specific return period, and the owners feel unsafe. Therefore, besides other possible retrofitting possibilities, structure/façade contractor of the movable façade system would submit a price quote and percent of dynamic behaviour improvement to the strata manager or building owner. The proposal and final submitted price quote would be based on the amount of construction work and renovation of the current façade system. Additionally, similar proposals can be offered for older existing buildings which use traditional stick façade system. In structures with traditional brick veneer façade system with opening window, the "In-plane" façade movement concept that was proposed in chapter three of the thesis can be used as one of the proposals. But this idea needs more research and validation before being proposed to industry and become commercial. Details of the proposal will be demonstrated as a future work in next chapter. The second scenario is for new building and price quote would be submitted to building developer and structural design team. The quote has proposals about possible façade systems that are capable of integrating with the new bracket system and the amount of improvement and financial saving regarding the dynamic behaviour of the future structure and structural materials (volume of pumped concrete and weight of steel) respectively. The cost of the smart façade system is definitely higher than the traditional facade system as the cost of the damper device and details of design or re-design would be added to price quote of the normal façade system. Table 7.1 shows the difference between prices of these two façade systems.

Table 7.1 Details of additional price of smart façade system

Item	*Assumed price(AU$)*	*Price per square meter of façade system (AU$/m2)*
Smart damper device	700 (per each bracket)	115
Details redesign	240 (per façade unit)	40

7.2.3 Maintenance

7.2.3.1 Preventive maintenance strategies and their cost

Several progressive degradation mechanisms begin to have a negative effect on the constructed new bracket materials. Therefore, to extend the service life of them, planned maintenance must be conducted. The maintenance strategies are mainly divided into preventive and corrective maintenance. Consideration of a regulated planned maintenance strategy is necessary at this stage. It should be mentioned that if the efficiency of the system is not considered and predicted at the design stage then maintainability issues will arise very soon. These external actions produce continuous deterioration in each one of the façade elements especially the ones located in the outer skin. The actions can only be avoided through a durable design method for the new system. Some maintenance strategies are proposed as below:

a) Various maintenance procedures need to be considered for implementation of the façade bracket dampers during its service life, including the necessary preventive and corrective maintenance.
b) The design process of the proposed system should be in a practical and efficient manner for the adapted maintenance strategy.
c) Maintenance techniques must be performed to guarantee the accomplishment of the design service life for every component of the facade system.
d) Maintenance operations should be facilitated by the adoption of a simple geometry for easy access for inspection of hidden parts of the damper facilities such as connecting steel joints.
e) Licenced architects or engineers who have sound knowledge of the design, material and construction of the bracket façade should conduct inspections. Periodical visual and detailed inspections must also be carried out on façade brackets to ensure the integrity and safety of the proposed systems and if inefficiencies are detected, corrective maintenance plans, by the type of deficiency, must be chosen.
f) Provide access for installation of any needed instruments for testing and developing information about the future behaviour and performance of the constructed materials and used components.
g) The building owner or strata manager should check with the design team to reach agreement on selecting the best façade cleaning method that is suitable for their building. Regular cleaning of the system for proper operation to avoid gradual development of deterioration sources (microclimates), especially during the summer season when severe conditions like high temperature and humidity are dominant. Managing these cleaning procedures to be performed from time to time is essential, as well as arranging guidelines for seasonal cleaning activities for critical zones that can accumulate salts and any other aggressive agents. The cleaning activities for the damper elements must be considered in preventive maintenance procedures to decrease the aggressiveness of the microclimates and these steps are as below:

- Cleaning of the expansion joints at the supports to clear the accumulated debris, dust and water-borne aggressive agents.

- Eliminating or cleaning the accumulated debris, dust and water carrying aggressive materials underneath the expansion joints.
- Cleaning the fixed and mobile bearings and to maintain them for suitable conditions to ensure their efficient and proper performance.

7.2.3.2 Quarterly and annual reporting of the proposed system

A team of professionals need to inspect the damper system at least once every three months to check overall performance of the system. Movement of materials, thermal movements, moisture movements, elastic deformations, creep effects and corrosion are the criteria need to be reviewed in the report. All of the deficiencies noted in quarterly reports should be documented and bundled in a yearly report. In this way, primary cause and level of severity of the issue can be recognised clearly. The severity of the identified deficiencies can be classified into the following conditions:

1. "Unsafe" is used when the identified deficiency causes a serious threat to individuals or property and should be immediately indicated to strata manager and local authorities by providing potential repair and corrective options.

2. "Requires repair/stabilization" is used for a case that may become unsafe if it is not scheduled for the next maintenance program.

3. "Ordinary maintenance" recognizes the cases when something is required to be addressed for the next scheduled inspection program.

Proposed spreadsheet which state condition of each damper component is shown in Table 7.2. This spreadsheet would assist the owner/strata manager with budgeting of future maintenance of the façade connectors based on their severity classification. The budget evaluation should include all costs of contractor's labour, materials, equipment, overhead, and general conditions, as well as the fees for architecture and engineering services, building administration, and unpredicted events. Spreadsheet for forecasted cost is listed in Table 7.3.

To guide the decision makers for planning the necessary repairs and future inspections, the report should include a comprehensive survey of history and condition of the façade panels in a way that can be understood by non-technical people as well. Additionally, the original building construction, alterations, renovations, and repairs should be included in making a more precise decision.

Table 7.2 Proposed quarterly and yearly spreadsheet for inspection of each damper/connector components

System components	*Severity classification (Inspected condition)*		
	Unsafe	*Requires repair/stabilization*	*Ordinary maintenance*
Rubber			
Steel layer			
Washer			
Visco-Elastic material			
Façade to bracket attachment			
Bracket to slab attachment			

Table 7.3 Spreadsheet for expected expenses per square meter of façade panel

Item	*Estimated cost (AU$)*
Contractor's labour	600
Materials	1100
Equipment	700
Overhead	300
Fees for architecture and engineering	1000
Building administration	500
Unpredicted events	20% of all expenses
Total cost	3840

7.2.4 Importance of thermal performance

One of the main criteria to evaluate the overall performance of a façade system as an outer skin of building structure is thermal performance. Maintain same thermal performance similar to conventional façade system is an essential element to convince the developers, building owns and insurance companies. Configuration and placement proposal of the damper system in the cavity between outer and inner layer is a crucial decision to minimize renovation expenses and quoted construction price for existing and new building structures respectively.

7.3 Building cost drivers

Cost is one of the first things that come to mind when a consultant, general contractor, or construction manager looks at a the potential of the additional component for the building. The design of a movable dissipative facade is the most significant step towards implementing a sustainable and durable façade system since it influences the whole service life of the structure, and it has a direct impact on its construction cost. The smart façade proposal should be logical enough with a comprehensive report on their performance to not to scares off the architect/client from using the innovative system in their building structure. However, it is important to understand that there are many factors that determine the price of the proposed system. This is important to have a system supplier on board early in the design process.

The overall construction cost of a building structure normally drives by many factors such as:

- Shape & geometry
- Size & regularity of floor plate
- Location
- Environmental forces(such as earthquake and wind) consideration in design
- Market conditions
- Seismic consideration
- Vertical transportation strategy
- Life cycle value
- Structural solution

Structural and non-structural Components which affect the overall cost are listed as below:

- Structure
- Curtain wall (conventional)
- Foundation
- Mechanical and electrical equipment (name as others)

In the Figures from 7.1 to 7.3 the average cost of a square meter GIA (Gross Internal Area) for buildings with respect to their height (low and high rise), location (London, Riyadh and Shanghai) and use (office or residential) are given, together with the breakdown among the major cost sources.

Once the target building has been defined with respect to the interests of the research investigation, the most relevant drivers for the financial analysis can be individuated. Obviously the most representative scale of the building for the investigation is the high-rise (office), but a characterisation of mid-rise type should be addressed as well. While high-rise building is generally affected by vortex-shedding from cross wind excitations, mid-rise building behavior is mainly driven by along wind phenomenon: two different test cases should be taken in to account.

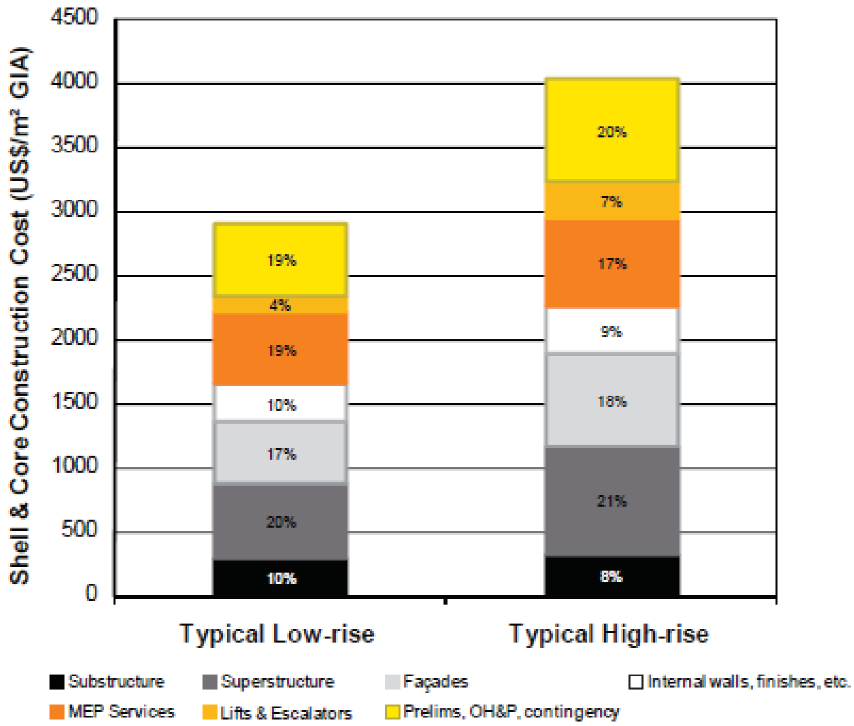


Figure 7.1 Relative elemental cost for Low and High rise office buildings in Central London [1]

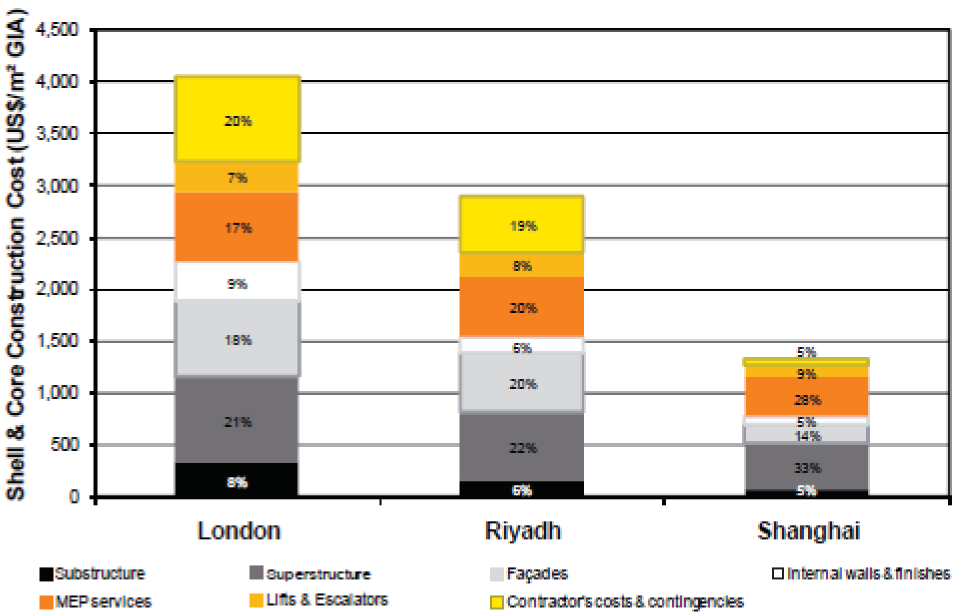


Figure 7.2 Typical elemental build up High-rise offices buildings in London, Middle East and Far East [1]

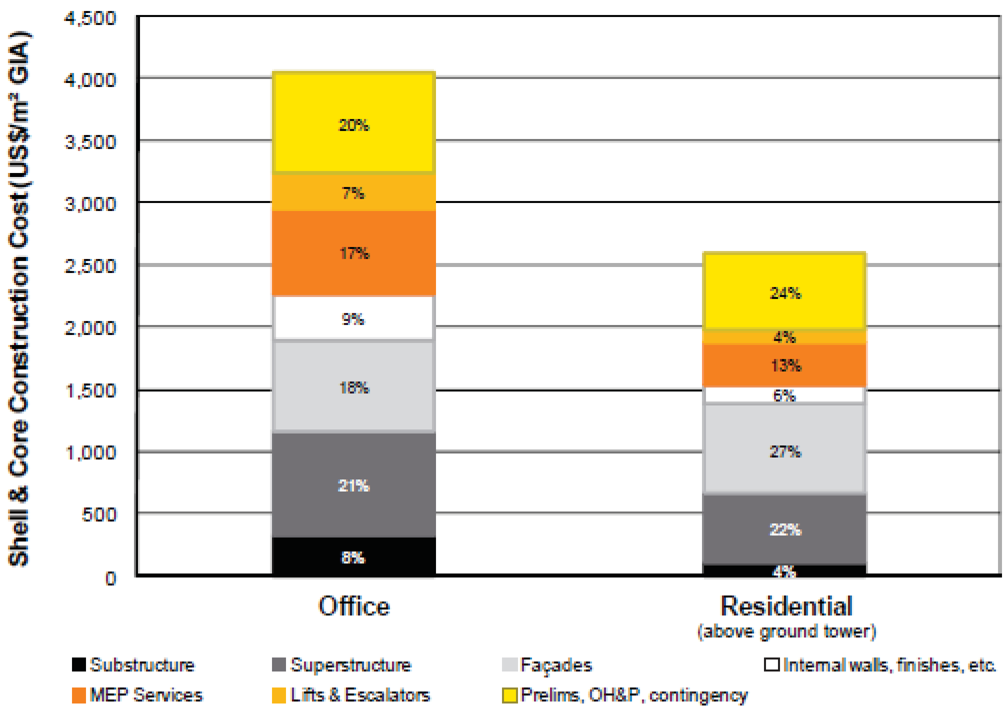


Figure 7.3 Typical shell and core construction costs: Office vs Residential Towers in London [1]

Moreover the dependence with respect to the market/location can't be neglected as it implies very large variations of the cost breakdown. This parameter will be discussed at the next section.

Shape and geometry are not be a matter of investigation and 'life cycle, Vertical transportation strategy' parameters which consider neutral from the target perspective are studied. Finally, structural frame design solution and design loading, and the façade cost will be discussed at the test case definition and investigated parameters sections, respectively.

7.4 Investigated Markets

The Case studies will be investigated for six different markets around the world, known for having several tall building in their landscape, as well as cities where Permasteelisa has significant market interests (Figure 7.4 and Table 7.4)

Figure 7.4 Case study: investigate markets

Table 7.4 Case study: investigated markets

markets	*Sydney*	*Shanghai*	*Tokyo*	*Dubai*	*London*	*New York*
Population (mil)	4.4	9.1	13.1	2.3	8.3	12.3
Mid-rise building (> 35 m)	845	1057	2779	568	1478	6504
High-rise building (> 150 m)	28	130	125	153	16	227
Mid/High rise building ratio	30	8	22	4	92	29
Mid/high rise per mil inhabitant	198	130	222	313	180	547

7.5 Investigated Parameters

This section is focused on the definition and the evaluation of the most important parameters for the assessment of the economic benefits derived by the façade damper use on buildings in comparison with the traditional solutions for the mitigation of wind loading effects.

7.5.1 Definitions

- The Gross External Area (GEA) is the area of the building measured externally at each floor level (Figure 7.5)

- The Gross Internal Area (GIA) is the area of a building measured to the internal face of the perimeter wall at each floor level(Figure 7.5)

- The Net Internal Area (NIA) is the useable area within a building measured to the internal face of the perimeter wall at each floor level with certain specified areas excluded (Figure 7.5)

The ratio between NIA and GIA has typical values for different sizes of buildings:

- 0.68-0.75 for low rise buildings
- 0.6-0.7 for high rise buildings

Other definitions typically adopted within building constructions:

- Class A buildings are most prestigious buildings in CBD for premier office users with rents above average. Buildings have high quality standards for finishes, systems, accessibility and a defined market presence. It is characterized by: central locations, first-class tenant improvements, on-site parking, state of the art elevators and HVAC systems, contemporary design and architecture, high quality of maintenance
- Net Rent is the average rent quoted per area per annum
- Gross Rent is the average rent quoted per area per annum and additional costs (property taxes, service charges, operation expenses)
- Cap Rate or capitalization rate is the ratio between the net operating income produced by an asset and its capital cost (the original price paid to buy the asset) or alternatively its current market value)

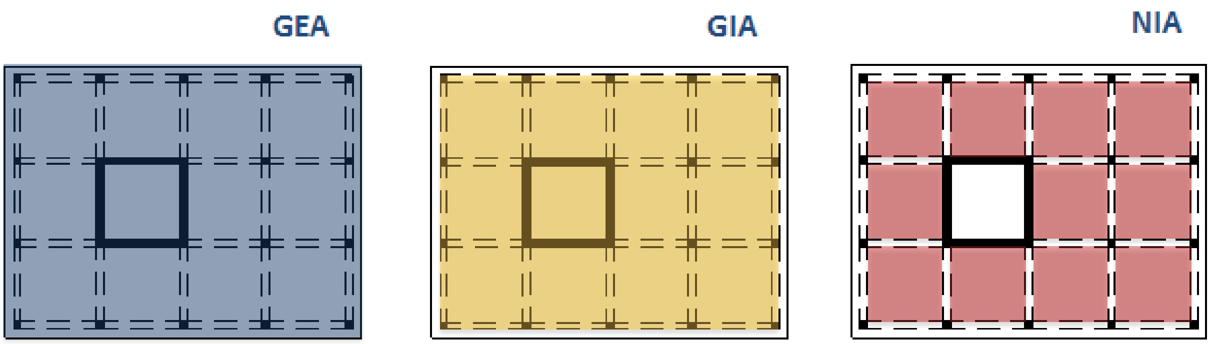


Figure 7.5 GEA, GIA and NIA definitions by a building plant example

7.5.2 Total construction costs

The average construction cost of Office class A building in CBD (in € per m^2 of GIA) are shown in the Table 7.5, with respect to the six cities selected [1,2]. The same values are then shown by the Figure 7.6, by means of absolute values and percentage breakdown of the several cost components. As already seen in section two of this chapter, London is the most expensive market, as the average construction cost is almost 5k€ per square meter of GIA. On the contrary, Shanghai, the representative town for the Chinese market, is the cheapest one with only 1.1k€ per square meter of GIA. Whilst the absolute values look significantly different, it can be seen the breakdown of the construction costs into the major shares has similar values for all the six cities. For instance the façade quote ranges in between the 14% in Shanghai and 20% in Dubai. On the other hand, the quote of the structure is dominant in Shanghai with around 33%, while it reduces significantly in London and Dubai, where the architectural and cost-effective energy requirements give more impact to façade and other quotes with respect to the structure.

Table 7.5 Investigated parameter – Construction cost per m^2 of offices (Class A)

Construction cost of Offices (Class A) in € per m^2 of GIA						
markets	*Sydney*	*Shanghai*	*Tokyo*	*Dubai*	*London*	*New York*
Foundation	205 €	66 €	291 €	85 €	389 €	175 €
Structure	640 €	364 €	669 €	311 €	1022 €	369 €
Facade	436 €	154 €	524 €	283 €	876 €	369 €
Others	1281 €	518 €	1426 €	736 €	2579 €	1028 €
Total	**2562 €**	**1102 €**	**2910 €**	**1415 €**	**4866 €**	**1940 €**

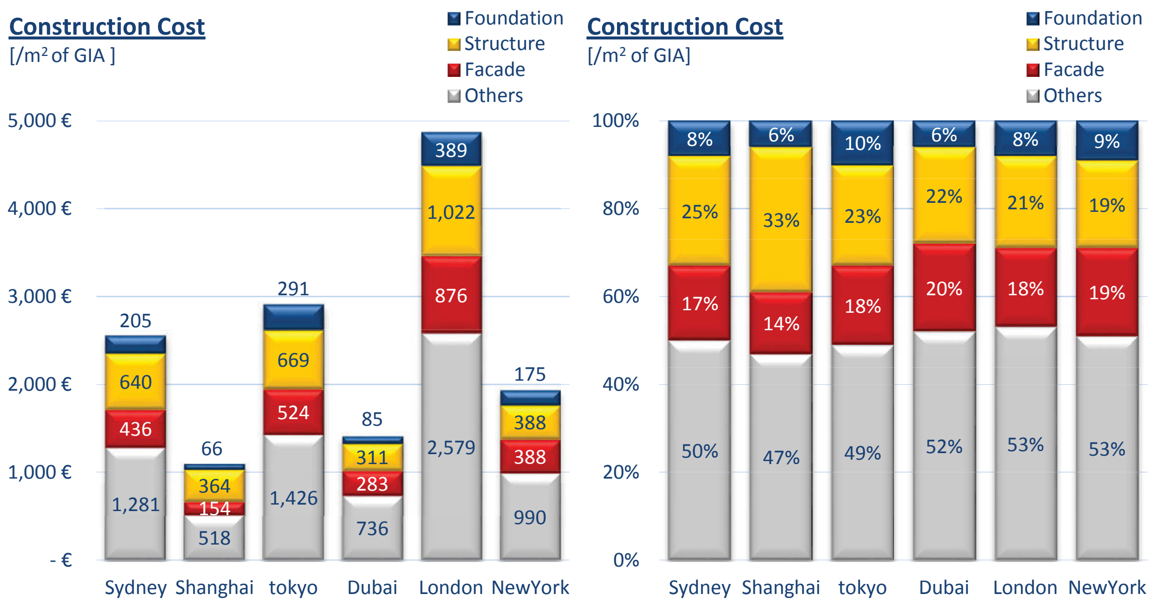


Figure 7.6 Investigated parameters – Construction cost and construction costs percentages

7.5.2.1 Material Costs

The material cost has been derived by the source [2]. For the major components, the following hypotheses have been made:

- Concrete: 30MPa of strength; job volume higher than 1500m^3
- Reinforcement: 16mm steel bar; job volume higher than 120 tonn
- Façade: curtain wall glazing including support system; job volume higher than 1000m^2

Table 7.6 shows the different unitary costs for the three components among the six selected markets. Then Figure 7.7 represents a bar comparison which highlights for instance the huge price difference of reinforcements between Sydney and Shanghai (almost double in Sydney) and the difference of the façade cost between Tokyo and Shanghai (almost five times larger in Tokyo). As the benefits related to the use of the façade damper involve savings on the material costs, it is important to characterize those benefits when applied to the different market scenarios.

Table 7.6 Investigated parameter – Material cost

markets		*Sydney*	*Shanghai*	*Tokyo*	*Dubai*	*London*	*NewYork*
Concrete	€/m^3	147	51	114	52	102	104
Reinforcement	€/t	990	533	632	608	919	764
Facade	€/m^2	833	189	958	293	704	597

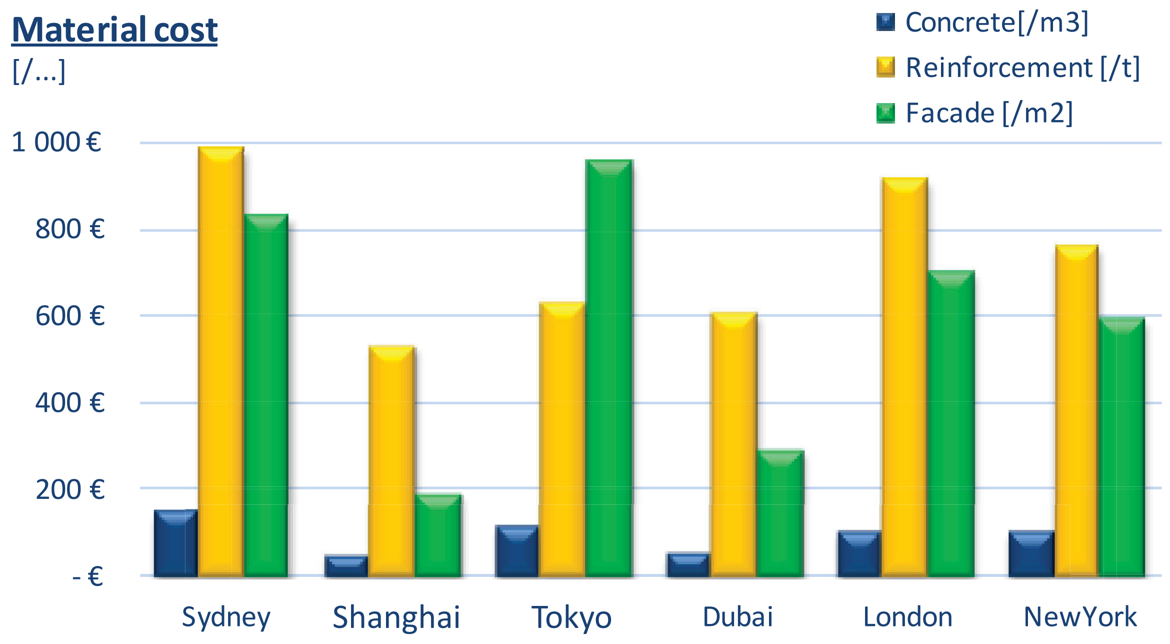


Figure 7.7Investigated markets – material costs

7.5.2.2 Labour Costs

The other important component of the construction cost is represented by the labour costs [2]. Table 7.7 and Figure 7.8 show the different cost for general and detailed workers among the six cities. It can be seen like the average costs are similar for London, Sydney and New York. On the contrary Shanghai and Dubai have really low labor costs.

Table 7.7 Investigated parameter – Labour cost, overhead included

Labour cost							
markets		*Sydney*	*Shanghai*	*Tokyo*	*Dubai*	*London*	*New York*
General	€/h	31	2	16	3	22	41
Builder	€/h	45	2	21	6	37	50
Site Foreman	€/h	58	5	23	15	57	59

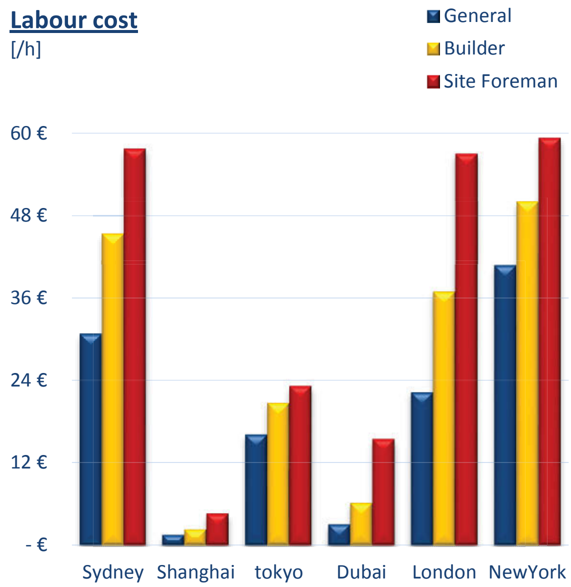


Figure 7.8Investigated parameter – labour cost

7.5.2.3 Construction time

In order to calculate the total construction costs due to the workers, the previous data must be compared with the needed construction time. The construction time is generally a complex matter, influenced by several sources of random errors [3]. The hypotheses made in order to perform the financial investigation are shown in the Table 7.8 and 7.9. In particular are included the consumption rates expressed in terms of workers per each working activity. Formulations for the calculations of the total time and total working costs by the input data are also visible from the Tables.

Figure 7.9 shows instead the expected effect of the façade damper with respect to the traditional façade in terms of construction costs. Two different approaches can be followed in order to assess the comparison: assuming a constant number of workers (then reducing the total construction time for the façade damper) or assuming a constant construction time (then reducing the number of workers). On both the cases the expected construction cost is reduced using the façade damper.

Table 7.8 Investigated parameter – Construction time

Costruction time			
A	Average labour consumption rate – formwork works	wh/m^2	0.65
B	Average formwork ratio for the entire building	m^2/m^3	From case study
C	Average labour consumption rate – reinforcement works	wh/t	8
D	Average reinforcement ratio for entire building	t/m^3	0.15
E	Average labor consumption rate – concrete works	wh/m^3	0.50
F	Maximum number of workers	wh/hr	From case study
G	Proportion of the average number of workers	%	80
H	Daily working time	hr/d	8
I	Concrete quantity	m^3	From case study
J	Buffer	%	10
NT	**Nominal time**	wh/m^3	A·B + C·D + E
NT	**Total time**	days	I / (F·G·H/NT) · (1+J/100)

Table 7.9 Investigated parameter – Construction cost

Construction cost			
K	Equipment and materials costs - formwork	€/m^2	From case study
L	Average formwork ratio for the entire building	m^2/m^3	From case study
M	Equipment and materials costs - reinforcement	€/t	500
N	Average reinforcement ratio for the entire building	t/m^3	0.15
O	Equipment and materials costs - concrete	€/m^3	From case study
P	Average wage	€/wh	From case study
Q	Mark-up for overheads	%	14
R	Buffer	%	10
NP	**Nominal Price**	€/m^3	K·L + M·N + O
TP	**Total Price**	€	I·(NT·P+NP)·(1+Q/100)·(1+R/100)

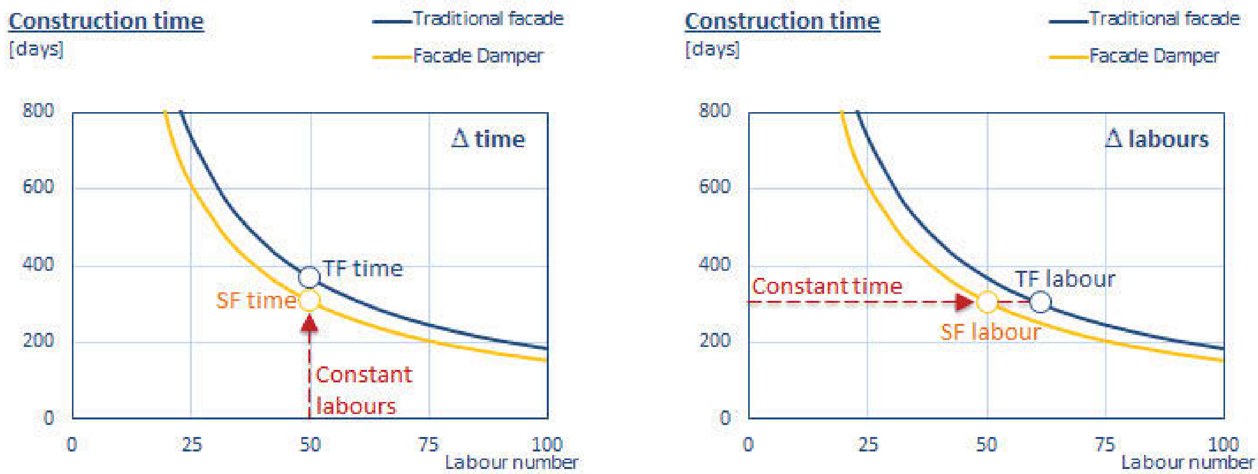


Figure 7.9 Expected construction time for traditional façade and façade damper by constant workers or constant time

7.5.3 Rental price and capitalization rate

The data for Table 7.10 and Figures 7.10 and 7.11 are taken from the source [4]. It can be noted that renting office building in Tokyo is highest follow by London and Sydney. Dubai has cheapest rent price between them. systems.

Table 7.10 Investigated parameter –Yearly Rental Price

Rental price								
markets		*Sydney*	*Shanghai*	*Tokyo*	*Dubai*	*London*	*NewYork*
Office *Class A; Net rent*	€/m² of NIA/y	524	394	738	316	655	308
Office *Class A; Gross rent*	€/m² of NIA/y	635	463	912	379	852	605
Cap rate	%	6.9	4.5	4.5	10.0	5.3	4.7

* London – City, ** New York – Midtown Manhattan

Figure 7.10Investigated parameter – Rental price

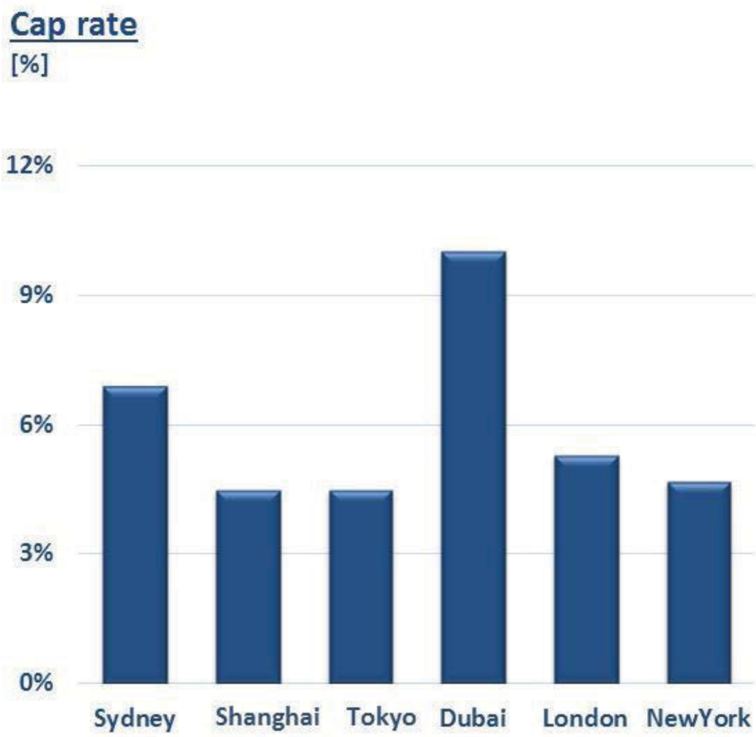


Figure 7.11 Investigated parameter – Capitalization rate

7.5.4 Damper façade price

The façade with the damper device has an additional cost with respect to the traditional one, which must be compared with the benefits derived by the savings on construction costs and rental income, in order to assess the damper façade business applicability.

Damper manufacturers have estimated series production costs for a damper, by means of a preliminary concept based on the input from technical results. The concept could be realized by an elastomeric damper device with hyper-elastic material model, as shown in Figure 7.12.

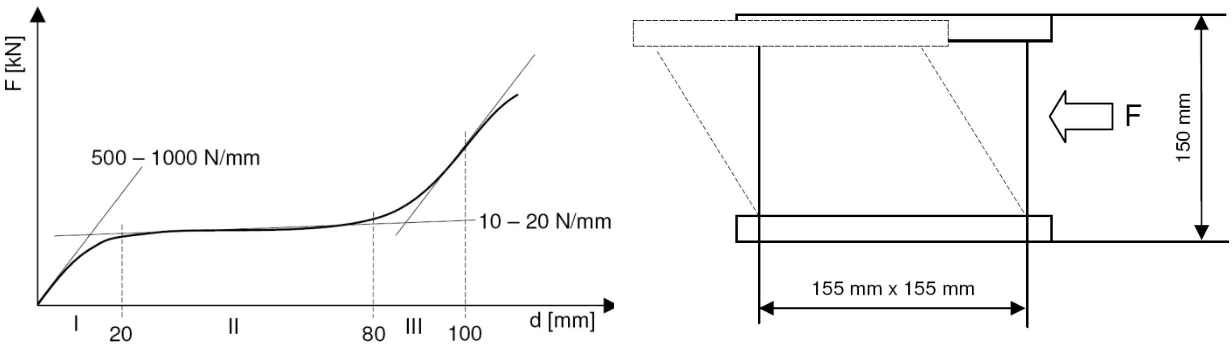


Figure 7.12 (a) Hyper-elastic material model (b) Façade damper device concept for wind

The assumed preliminary price of such device is 450€/piece (minimum 400pieces). Assuming a standard façade unit size of 4mx1.5m=6m^2 and single devices per façade unit, the additional price of the façade directly due to the damper component can be assumed to be 450/6=75€/m^2. However additional cost for re-design of the typical façade-component must be considered as well, estimated in 25€/m^2. Then in total the cost of the damper façade can be estimated in cost of the traditional façade plus 100€/m^2. Figure 7.13 shows the schematic view of the smart damper and the bracket which is going to carry the vertical loads (weight of panels and damper) and horizontal force (wind force).

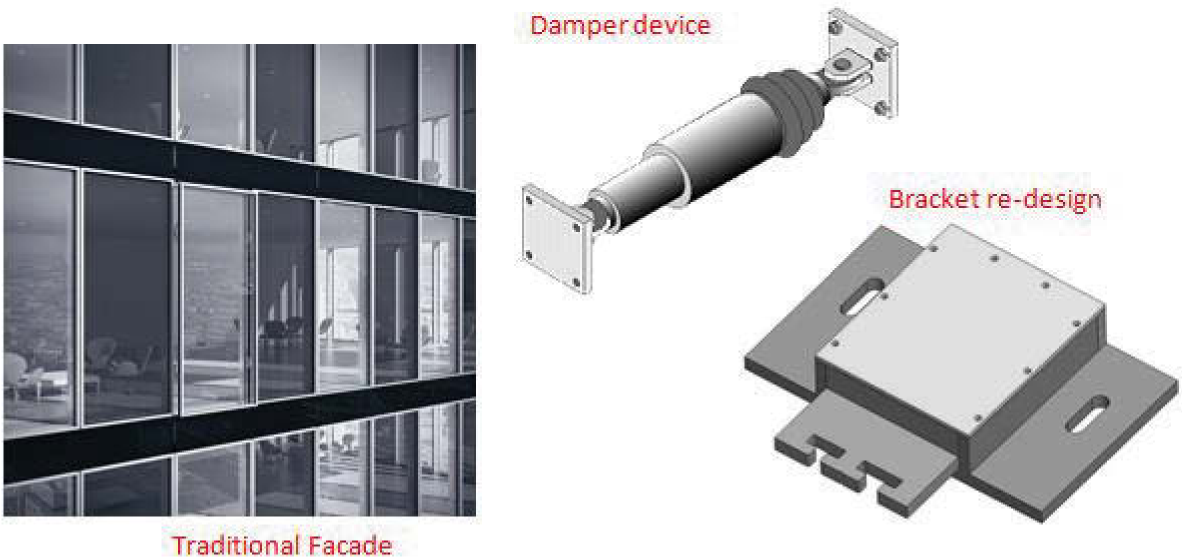


Figure 7.13 Damper façade as traditional façade combined with damper device and new bracket design

7.6 Test case definition

The Figures 7.14 and 7.15 show the floor plan and the major input data for the definition of the mid-rise and high-rise test case buildings.

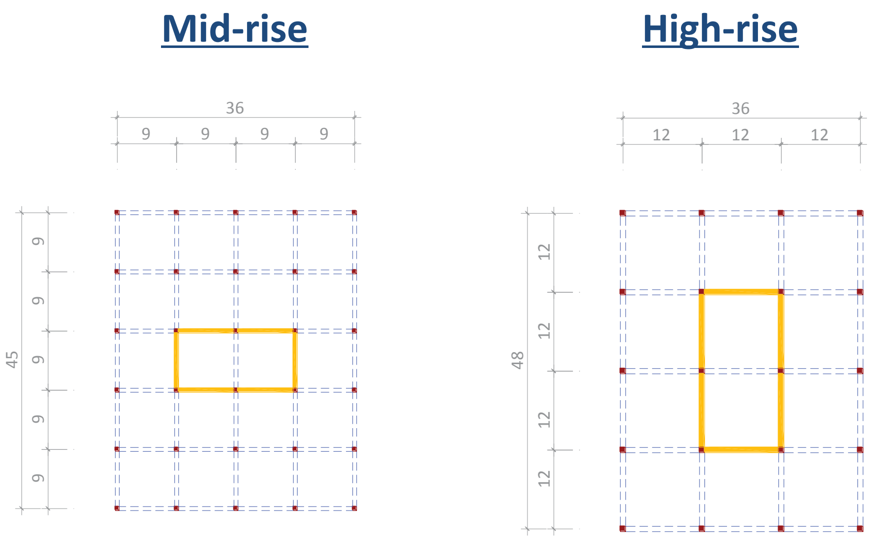


Figure 7.14Mid-rise and high-rise test case building floor plant

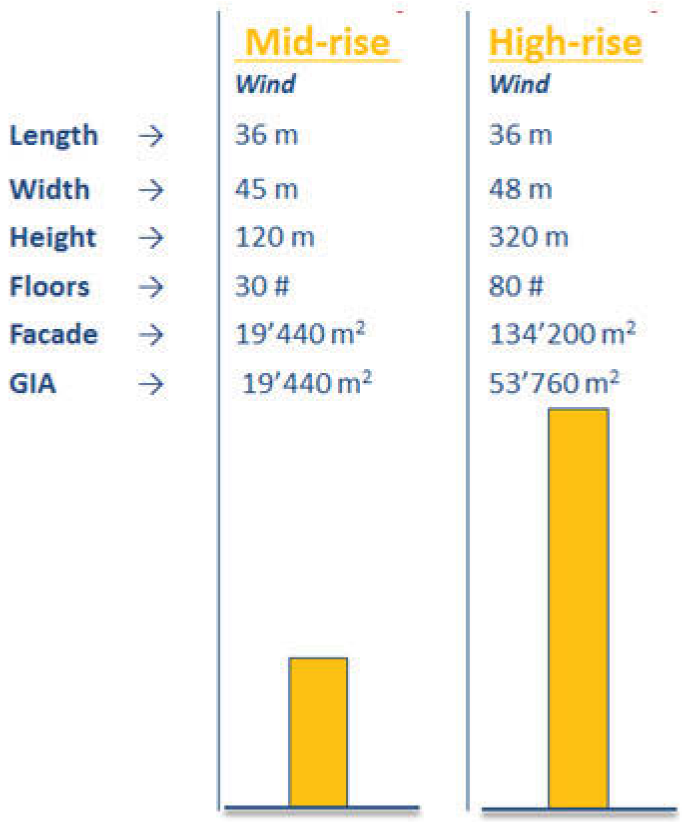


Figure 7.15 Mid-rise and high-rise major input data

7.6.1 Comparative approach

The comparative procedure for the financial assessment of the damper façade can be summarized by the following steps:

- Wind speed design time history definition (Figure 7.16)
- Design of typical building with traditional façade against lateral load (5% of the dead load)
- Assessment of typical building (with traditional façade) performance under yearly wind loading. Typically the building experiences more than 10mG of peak acceleration.
- Assessment of typical building (with damper façade) performance under yearly wind loading (Figure 7.17). Typically the building has less than 10mG of peak acceleration.
- Enhancement of the typical building (with traditional façade) in order to match the performance of the traditional building with damper façade

Once the typical building with façade damper and the enhanced building with traditional façade have been characterized in terms of frame structure cross sections, the economic

assessment can be conducted and the possible savings can be calculated. Total cost of the buildings can be assessed considering material costs, construction time and labour work.
In addition to the possible savings on the construction costs, the additional rental income due to the gained rental space must be considered in order to evaluate the economic benefit of the damper façade.

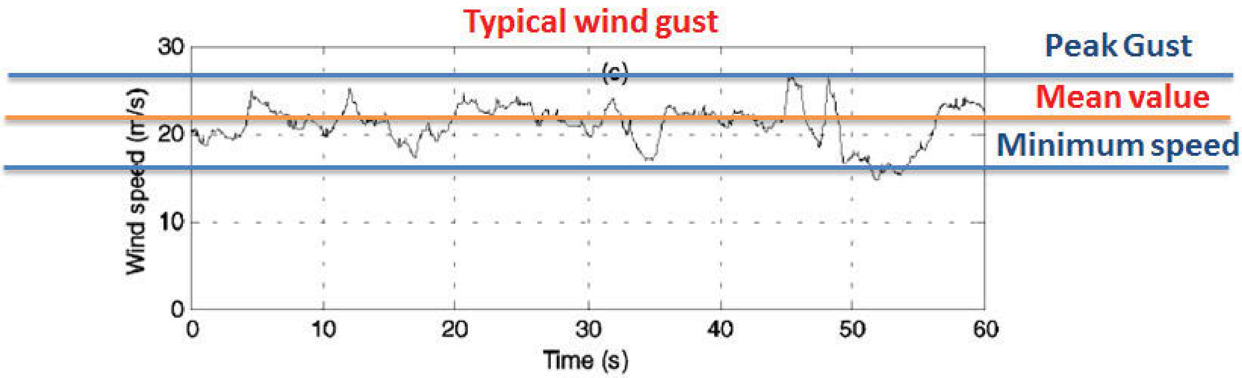


Figure 7.16 Definition of the wind speed design time-history

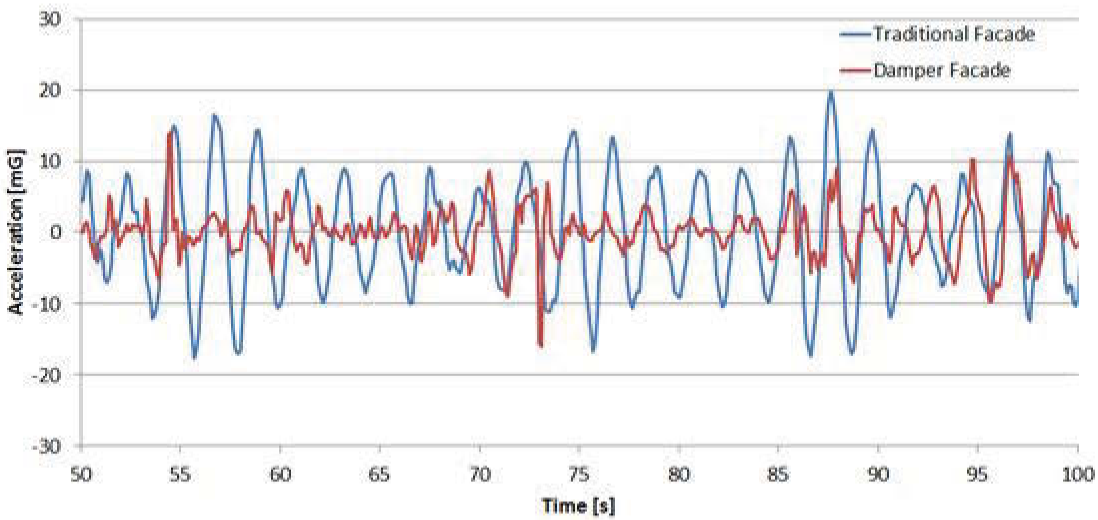


Figure 7.17Typical comparisons for acceleration at the top of the building with traditional and damper façade

7.7 Case Study Results

This chapter includes the major outcomes of the economic assessment for the mid-rise and the high-rise test cases defined at the previous section.

7.7.1 Mid-rise Building Results

Figure 7.18 shows the comparison between the cross sections designed in order to comply with the acceleration criteria for the building with conventional façade and for the building with damper (smart) façade.

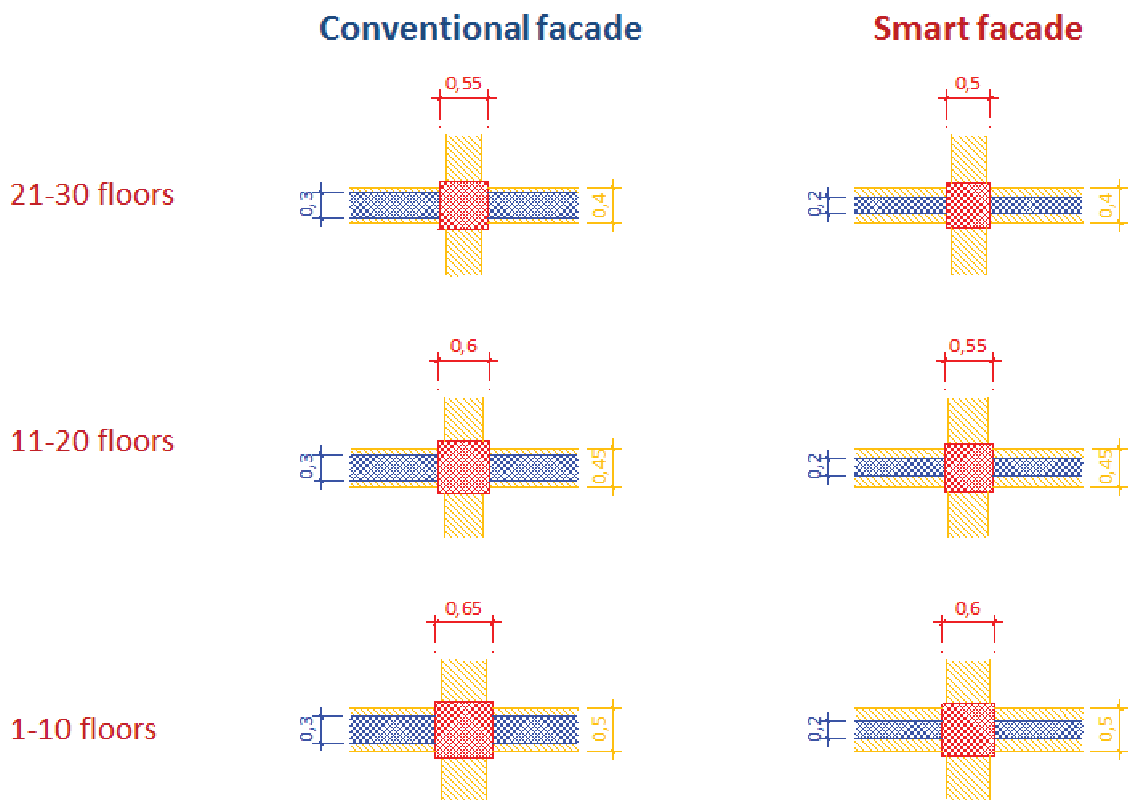


Figure 7.18 Building frame design with conventional façade and with damper (Smart) façade

Figure 7.19 Construction costs for Conventional Façade and damper (Smart) Façade (Sydney)

On the basis of the previous differences in terms of cross section area, the Figure 7.19 shows the breakdown of the construction costs for the conventional façade and for the damper façade in Sydney. The percentage of each cost quote with respect to the total construction costs of the conventional façade building in Sydney are shown in Figure 7.20, which highlights how the savings corresponds to the 0.7% when the façade damper is used. Figure 7.21 displays instead the reduction of construction time and labour costs.

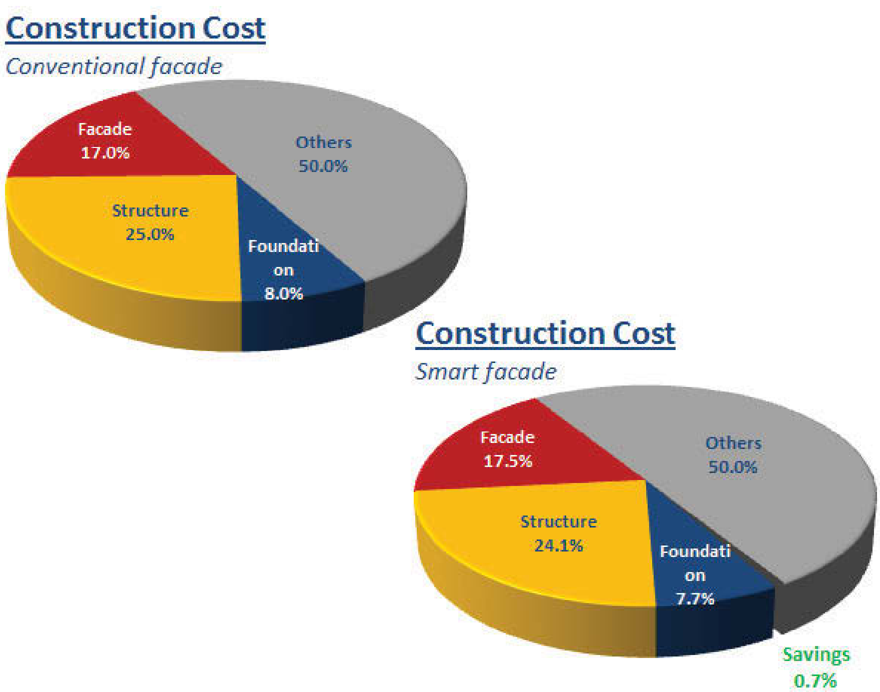


Figure 7.20 Saving percentage on construction costs (Sydney)

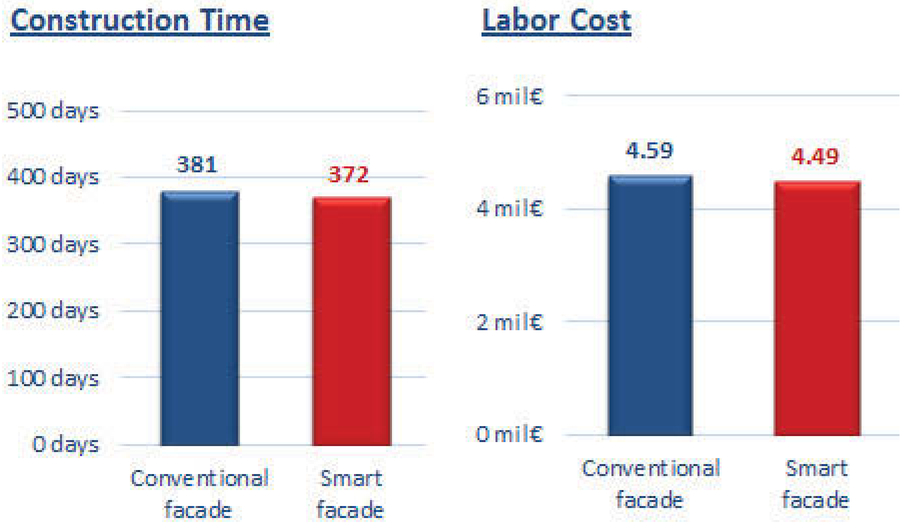


Figure 7.21 Construction time and labour cost savings (Sydney)

Figure 7.22 represents the major beneficial effect due to the damper façade. Because of earlier entrance and saved space, the potential income for the building owner increases. The overview of the costs and rental income along the six selected markets is shown in Figure 7.23.

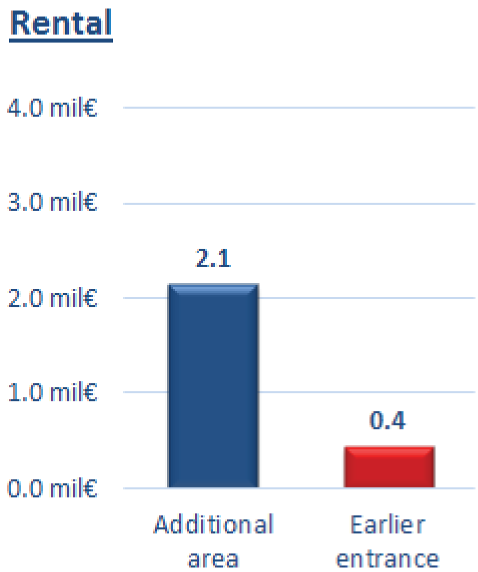


Figure 7.22 Additional incomes from additional area and earlier entrance (Sydney)

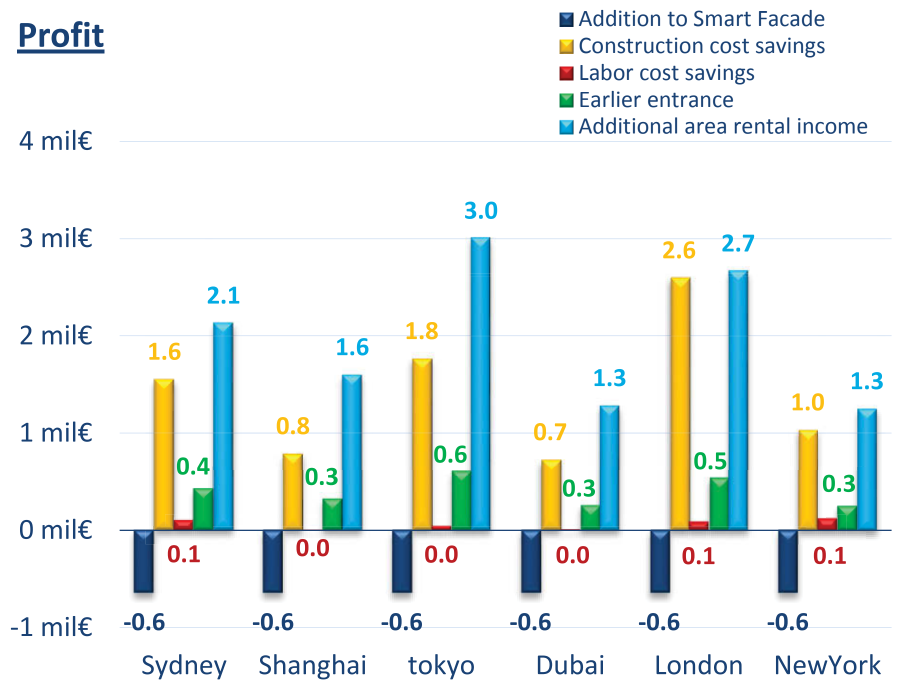


Figure 7.23 Profit breakdown of the damper façade against the conventional façade versus the six selected cities

Figure 7.24Total profit of the damper façade against the conventional façade versus the six selected cities

Figure 7.24 summarizes the combined effects of the breakdown shown in Figure 7.22. As it can be seen, the damper façade would offer a significant total potential margin to the owner (obtained summing earlier entrance, rental income and construction cost savings) in all the six cities investigated.

7.7.2 High-rise Building Results

Figure 7.25 shows the comparison between the cross sections designed in order to comply with the acceleration criteria for the building with conventional façade and for the building with damper (smart) façade. By means of those differences, the breakdown of figure 7.26 and the summary of figure 7.27 can be found for the high-rise building test case.

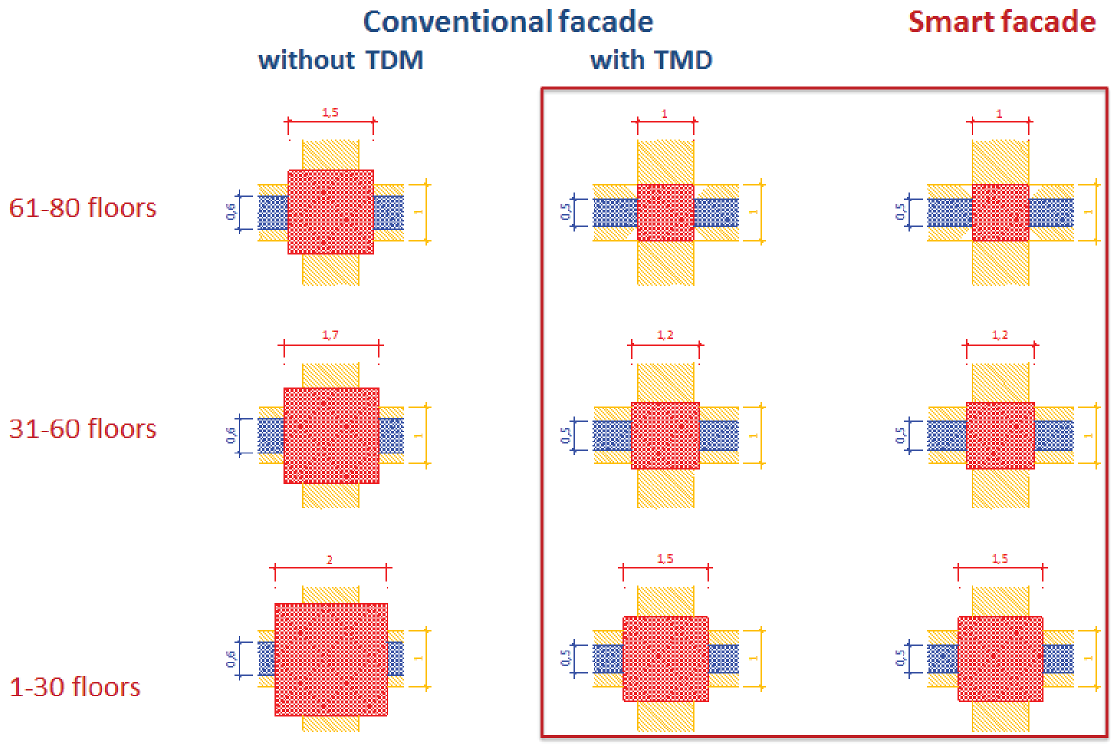


Figure 7.25Building frame design with conventional façade and with damper (Smart) façade

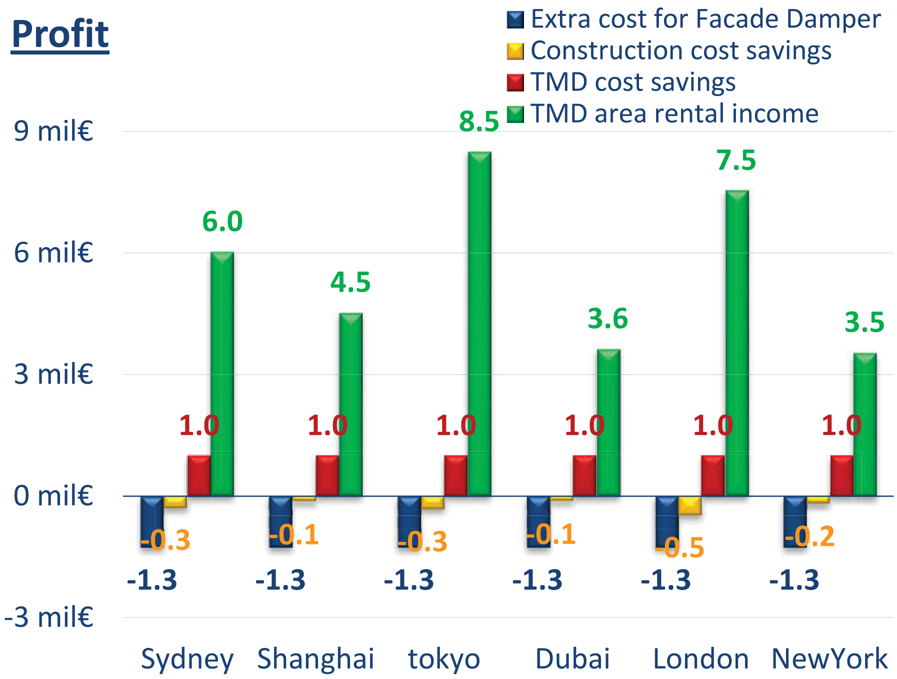


Figure 7.26 Profit breakdown of the damper (Smart) façade against the conventional façade versus the six selected cities

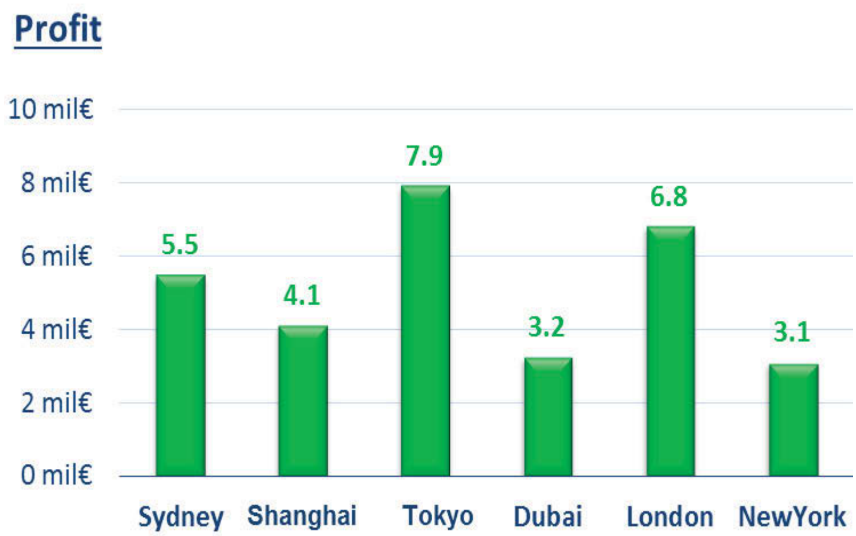


Figure 7.27 Total profit of the damper façade against the conventional façade versus the six selected cities

7.8 Summary and conclusion

The design principle for the smart facade system was proven through extensive numerical analysis. The proposed system like any other material will degrade and lose their functionality over time. Despite the fact that, deterioration of façade materials is an unavoidable phenomenon, but service life of the damper material can be extended to an optimum value by considering possible durability issues in the stages of design, construction, operation, maintenance and repair. These stages are discussed briefly as below:

1. Design:

Design of the system should be in a way that steel layers of the damper system have minimum contact to open air and surrounding environmental. Based on various climates and region in which the damper is going to be installed, different strategies need to be considered in the design stage to avoid any deterioration that is caused by environmental factors. The designer must focus on detailing of the system in a way to minimize the probability of untimely degradation of the components. Finally, effects of criteria includings resisting structural and environmental loads, heat and air transfer, preventing water and moisture infiltration, acoustics should be considered in façade design.

2. Construction

The damper layers need to be ordered from 3M company which is one of the few companies in the world that can make the delicate viscoelastic damper layers. Construction and assembly of the damper system would be in workshop for lower quantities or in a factory for mass production depending on budget of the project and amount of external investment. For that reason, supervision by a specialist is quite essential during construction activities to ensure that the installation work is conducted in line with the original design and specifications. Depending on types of structure namely as existing or new building structure, a careful attention needs to be considered during installation process of the façade damper systems. High standard of workmanship is needed to minimize the deteriorations involved in poor installation of the panels

3. Operation and maintenance

The higher is the quality of the layers the more durable is the system. For that reason, regular quality control is paramount during service life of the system to ensure that performance of the system works is conducted in line with the original design and specifications. Inspection need to be done by fully trained technician under supervisor of façade/material experts. The manager should keep a record of the results of all inspections throughout the service life of the damper system and should assess the safety of street-level facilities by consulting these records. Any deteriorating salts or ions need to be removed promptly to enable the assessors to perform a better visual inspection. If serious deteriorations take place before the end of the service life, and no maintenance work is conducted to correct the deterioration, the system may lose its functionality and cause serious damage. Corrective maintenance would be needed, only if a detailed inspection reveals that rehabilitation work is needed. Therefore, planned maintenance must be performed to ensure safety and serviceability of the whole system.

4. Repair

In the absence of regular maintenance, the façade bracket elements would continue to deteriorate, and finally its repair/ rehabilitation could become expensive, requiring replacement of the damper system. A specialist needs to consultant with the designer and decide the best rehabilitation process for restoring the bracket damper to its original condition. Replacement of components is very necessary at the end of their service life to ensure maximum dynamic performance. It should say that, the module width and height, number of points of support, glass make-up (thickness and performance) required, back-up structure, are all important factors which depend on need of the clients and would change in each smart façade system contract. System pricing can vary greatly depending on requirements. The facade inspector should be experienced in the field of stability, defect deterioration, forensic investigation, remedial engineering mechanisms and expert witness relating to specific materials and facade assembly. The design team can advise building owners of potential cost savings measures by modifications in design if there is a budget range in mind. Post-pos-wind repair costs and repair time are two crucial parameters, which need more research. Because, detachment of façade component from the building which may lead to social and economic damages as well as increase in injury or death to the occupants and pedestrians. Smart façade system, if design and install properly, has better economy of scale on a cost per square meter basis.

Using of smart façade system has these benefits and advantages to the building structure in compare to existing retrofit technics.

1. rental area

   Rental area losses permanently in current retrofit technics although there would be no loss in the rental area when smart façade system is used to enhance dynamic response of main structure.

2. windows or/and doors

No closing or opening process is needed in smart façade panels.

3. Time

No temporary losing of rental income happens in smart façade system, although tenants should move out of the offices when current retrofit technics is selected.

4. Re-design cost

No structural modification is needed in the building structure when smart façade system is selected as a method of retrofit.

Chapter 8

8 Conclusions and Recommendations for Future Research

8.1 General Conclusion

Modern architecture was created based on modern technology and grew into a new architectural style that was preferred internationally. Tall buildings, which emerged in Chicago in the late 19th century, even before various architectural movements in Europe, are clearly forerunners of the mainstream modern architecture of the 20th century. This new building type was created through the development of iron/steel braced frames and curtain wall concepts. Today, in this intellectually and culturally pluralistic era, architecture is still deeply rooted in these original technologies, which enabled early tall buildings/modern architecture to prosper as a new style.

In tall buildings, which in a sense are the accumulation of the most advanced architectural technologies, the significance of integrative design approaches is more important than any other building type to overcome or at least minimize contemporary technological limitations. Design integration requires intimate collaboration between architects and engineers. Understanding the technology, science, and mathematics behind the behaviour of the system is the responsibility of contemporary architects who want to create higher quality architecture. Likewise, understanding fundamental architectural design principles is the responsibility of engineers who want to achieve higher quality engineering products that are incorporated into architecture. This mutual understanding can become the potentiality of enhanced design integration.

The study of movable double skin façade, a new direction for solving the motion problem of tall buildings was introduced. The studies on the theory, preliminary design guidelines, and architectural implications of distributed tuned mass dampers will be useful for both architects and engineers. Within todays globally prevalent architectural context, which values energy efficient design approaches more than ever, integrating structural motion control with double skin façade systems has great potential.

Double skin façades in tall buildings is one of the most advanced forms of façade systems available today. This study have been investigated the other potential functional of double skin façades in tall buildings as lateral motion control devices. The new Damper façade System (DFS) design utilises the non-linear hysteresis phenomenon associated with nonlinear dampers for the façade connections between inner and outer skins. The system is controlled by a passive behaviour damper and it will activate when the outer skin façade subjected to design wind conditions.

The outcome of this study show that using façade as a control system is feasible. Using the outer skin to filter input energy has significant effects on the response of the primary structure. Previous research shows that this system has potential to dissipate the energy, but requires a very large façade movement which is not practical. This research represents a unique solution to make movable façade practical. By controlling the connector stiffness and introducing variable stiffness, one could reduce the primary structure response and also limit the movement of the outer skin of the façade to a practical value.

Investigations have illustrated that best results, in terms of ductility reduction, can be achieved using a façade moving at the TMD frequency (bracket with low stiffness). On the other hand, façade brackets should be designed to withstand strong winds, it means that they should have reasonable vertical and horizontal displacements; otherwise, they may go out of alignment and pose danger to occupants and pedestrians (falling down objects). It should be noted that, based on façade design calculations, each façade panel should have a plus/minus reasonable displacement during the application of forces.

The main concept is to replace the conventional bracket elements, which have rigid behaviour, with new bracket elements with weak axial stiffness to filter the input energy in case of wind. The proposed behaviour of bracket elements is shown in figure 8.1.

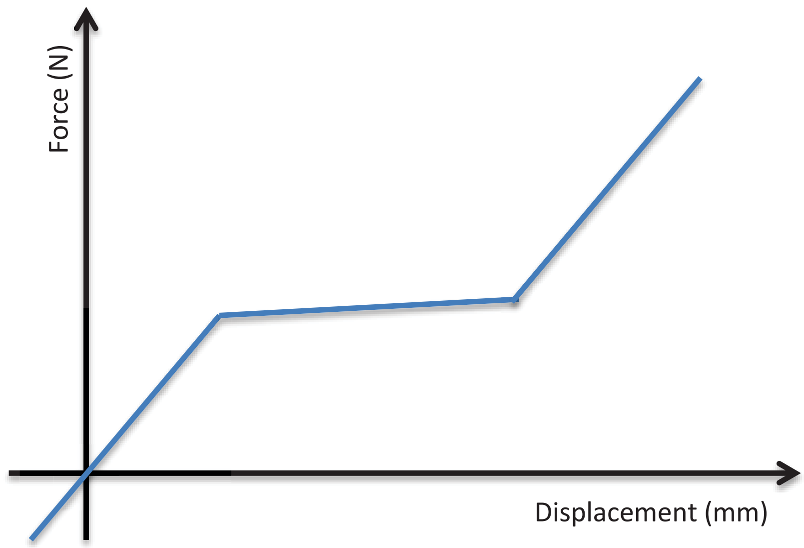


Figure 8.1 Multi-linear behaviour

To conclude, the main findings of the study are presented as below:

1. It is feasible to use energy-absorbing connections in building facade system to control response of main structure by dissipating part of applied force under wind loads. Viscoelastic damper have proved to be very efficient for this purpose and the connection properties have significant influence in the response. They have optimum values of stiffness and damping based on intensity and kind of mean wind speed values.
2. The energy absorbing connections placed in direction perpendicular to applied wind force were able to control the deformation and forces in structural elements with reasonable differential displacement between frame and façade.
3. Influence of façade mass on the wind response was investigated as well. Results showed that increase in the mass ratio resulted in higher reductions in differential building response. However, the increase in the mass ratio should be in a control way and logical.
4. Controlling the response of the building system was possible even when the natural frequencies of the structure were within the range of dominant frequencies of the mean wind speed.
5. The optimum façade damper stiffness value in second path is 0.2N/mm-0.5 N/mm which can be effective in reduction of response of main structure in both Mid-rise and high-rise building structures. As it is hard to install bracket elements with different value of compression stiffness because of complexity of their design, then implementing a semi-active or active control system is needed for the proposed concept.

Overall, the use of the comprehensive time-history analysis in Chapter 4 and 5 and bracket design in Chapter 6 also financial assessment of the proposed system in chapter 7 provide a rational framework for selecting appropriate stiffness for designing the cladding in various geotechnical areas.

8.2 Application and contribution of this research to design

Double-skin Facades have made a rapid dispersal into the commercial markets such as Australia, North America and specially Middle East. In wind case analysis and design, structural engineers typically disregarded the extra stiffness and damping that the cladding system may add, which could demonstrate to be beneficial to the building's wind performance. The study has indicated the possibility of developing new façade connections with appropriate properties to reduce response of main structure during gusty wind. The idea of advanced cladding connections developed in this research was created to take advantage of energy dissipation due to the relative movement of the cladding panels and structural frame.

However, these systems necessitate cladding systems to encounter significant lateral movements to create any promising effects; therefore, crucial criteria such as the appearance, water tightness, and air tightness due to the relative panel-structure movement could be threatened. On the other hand, From the several time-history analyses carried out, it was evident that with the implementation of appropriate connection properties, the gust wind response of the main structure can be considerably reduced. Moreover, the connection deformation and the connection forces can be kept within reasonable and practical limits by applying pre-defined load deformation behaviour to them. Engineers, often, would prefer to optimize a system for a particular property such as low energy consumption. Particularly in the design of buildings, it is difficult because of conflicting priorities such as optimizing daylight and minimizing solar gain. Thus, co-optimizing is essential. The link between architects, engineers, and facility managers must be managed carefully to develop and use complicated control features that do not overwhelm users or the lack of management may render the advanced facades inefficient. Additional design tools must be developed so that structural and façade designers may easily investigate effects of movable double-skin facades on seismic performance of the structure during design stage. It is noteworthy to mention that, acceptance of the proposed façade system is linked to the additional expense to the building owner also architectural and environmental benefits indicated on their behalf.

8.3 Recommendation for future research

The research discussed in this dissertation has focused to answer the important questions related to the wind capability of movable cladding systems especially double skin façade systems in multistorey buildings. However, more research in the field will provide more comprehensive results in terms of structural geometries, cladding configurations, and connection types, also can help code committees to revise and add a section for designing movable dissipative façade system in wind prone zone areas. The data used for these analyses is based on analytical and very limited experimental tests on movable cladding system components.

Thus, to improve the performance of the proposed system, more experimental tests on existing and new cladding system will provide statistical data analyses for design handbook for various types of cladding connectors typically found in construction. In addition, more discussions with cladding manufacturers and contractors will provide additional data and give general overview on the system and corresponding different repair methods. The physical test will provide not only a comprehensive overview of the proposed system to, but also give valuable data to re-calibrate the properties of the damper material in ANSYS models. Determine repair costs and repair time of the proposed cladding system are necessary part of future experimental work in order to fully evaluate the practicality of the system. More research on the structural response of non-regular structures with different height and plan irregularities in three-dimensional is a crucial need for completing this journey.

8.4 Further research that would improve and complement this thesis

It has been observed from the contents of this research that the focus of this PhD is on numerical modelling and proofing of the proposed concept. Further tasks need to be considered are listed as below:

a. Façade panel distortion, local and general deformation in connections need to be looked at

b. Micro modelling of the proposed attachment which is shown Figure 8.2 needed. The modelling needs to be sketched in Solid work program and export to ANSYS or ABAQUS for further and accurate analysis.

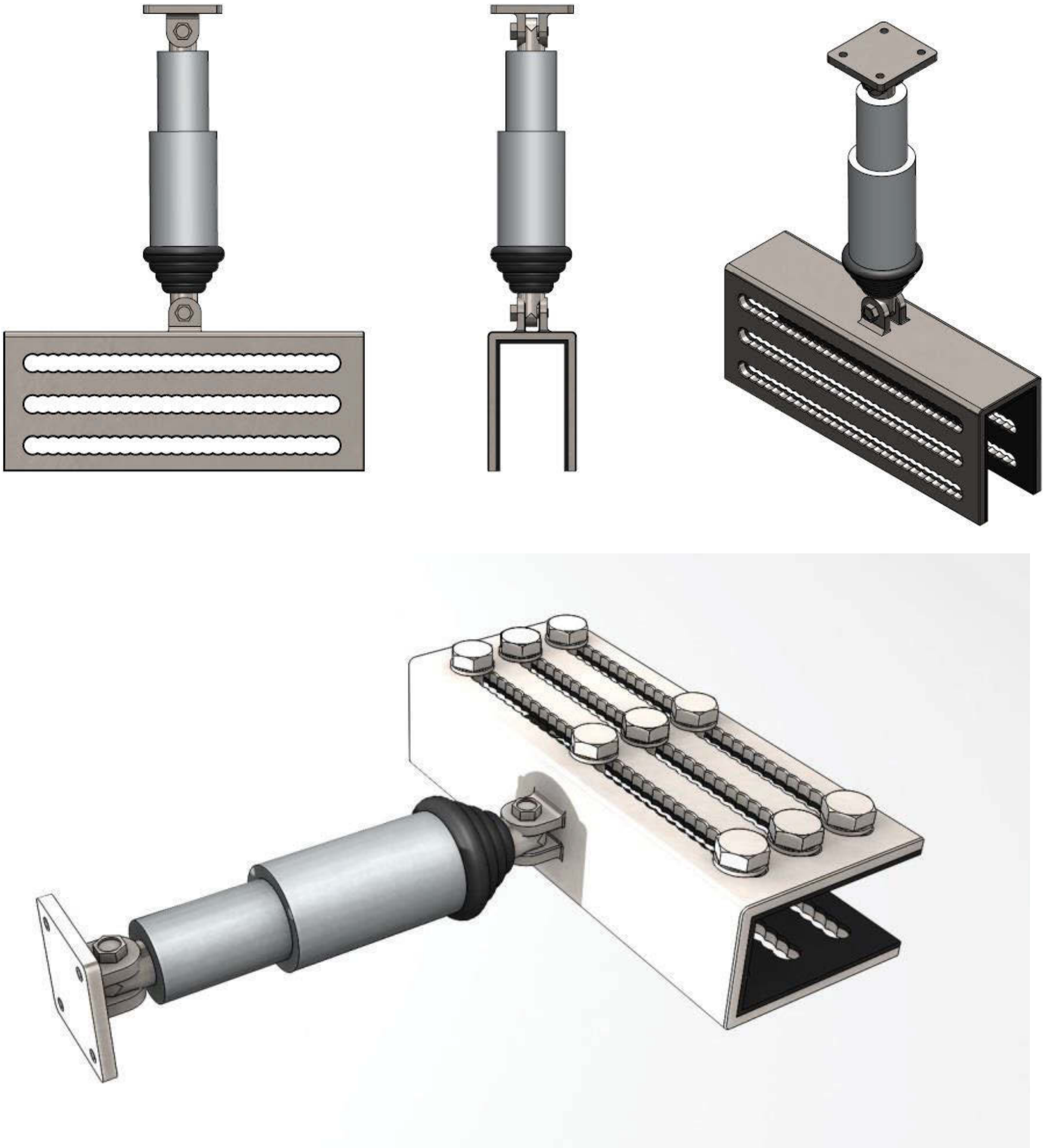


Figure 8.2 Details of proposed connection for attachment of façade outer skin to slab of main structure

c. Gap width between outer and inner façade layers need to be optimized. The random variable consists of length of axial damper element need to be evaluated numerically and experimentally.

Proposed experimental test program

The major part of the research in this dissertation focused on computer simulations of cladding systems and reliability studies. To gain additional insight on the seismic performance of multistorey buildings with movable cladding system, an experimental test needs to be performed. The proposed experimental testing program will provide insight into the three dimensional behaviour of the movable cladding system. This section outlines a proposed testing program to evaluate the response of a full-scale portion of a movable cladding system. The goals of the tests are to understand how the cladding system components interact with main structure and understand their dynamic behaviour. Additionally, their interaction together as a uniform layer is very crucial for possible collision and internal damage and needs to be monitored carefully. The results of the tests should provide some validation to the analytical results presented in this dissertation. The simulation needs to provide insight into how panels and connectors behave during a wind flow along the entire height of the building and to validate the analytical models developed in ANSYS APDL and also ANSYS Fluid. Locations of possible damage and identification of the failure modes of the connections need to be determined as well.

Test setup, specimen design and terminology

The building is indicative of moment frame structures common in commercial real estate. Although the material of the building structure was reinforced concrete in the numerical analysis, but in order to have easier set up and installation, a steel structure frame is going to be built with pinned connections so that the frame has less lateral resistance. The beams and columns are considered box sections. A sketch of the south-west elevation of a corner specimen is illustrated in Figure 8.3. Two panels which are in red colour on top level of building structure are selected for experimental test analysis. They can attach to a main frame individually or ideally together to evaluate their interaction with primary structure and each other. Detail of proposed experimental test model in different angles is shown in Figure 8.4.

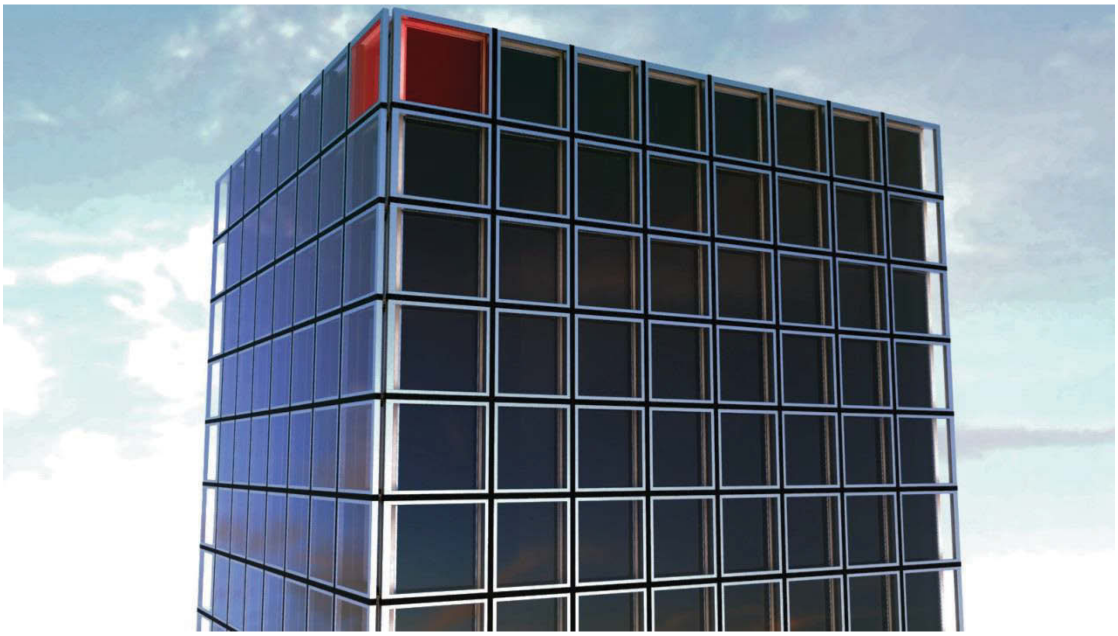


Figure 8.3 South-west sketch of the building structure and elevation of the specimen

Although the test specimen does not represent all kinds of the cladding panels on the high-rise and the mid-rise structure buildings, but the full-scale corner specimen provides a unique opportunity to evaluate the interaction between the movable facade panels as the frame moves. Corner subassembly experience largest inter-storey drift and largest post-yield drifts during wind loading. Additionally, dynamic response and panel interaction at the corners of the building structure are difficult to understand and it would be a good opportunity to evaluate this interaction. The façade system, connection types, and connection locations need to be considered same as discussed in this dissertation. Full-scale cladding assemblies measuring 3600mm (one story) tall by 1500mm wide will be tested to investigate the interaction of the cladding panels in plane and out of the plane. Two quasi-statically tested specimens are expected to gather information about the overall behaviour of the system, connector's behaviour, the interaction of façade panels at the corners, and the progression of damage in dissipative damper devise.

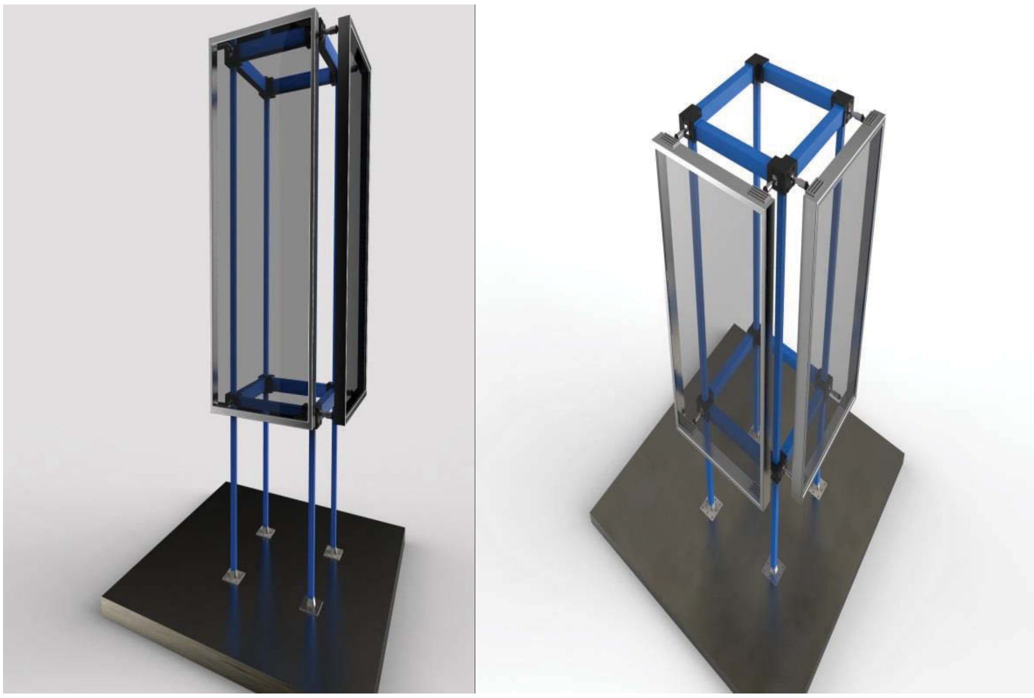


Figure 8.4 Sketch details of experimental model

Appendix A

Mid-Rise Structure 2D Analysis

In figure A.1 the corresponding acceleration versus time is outlined regarding to the wind with a mean value of 15m/s. Due to the increase in the mean wind speed by 3m/s, some beneficial effects are expected. As shown in figure A.1, the smart façade shows better response as compared with the previous status.

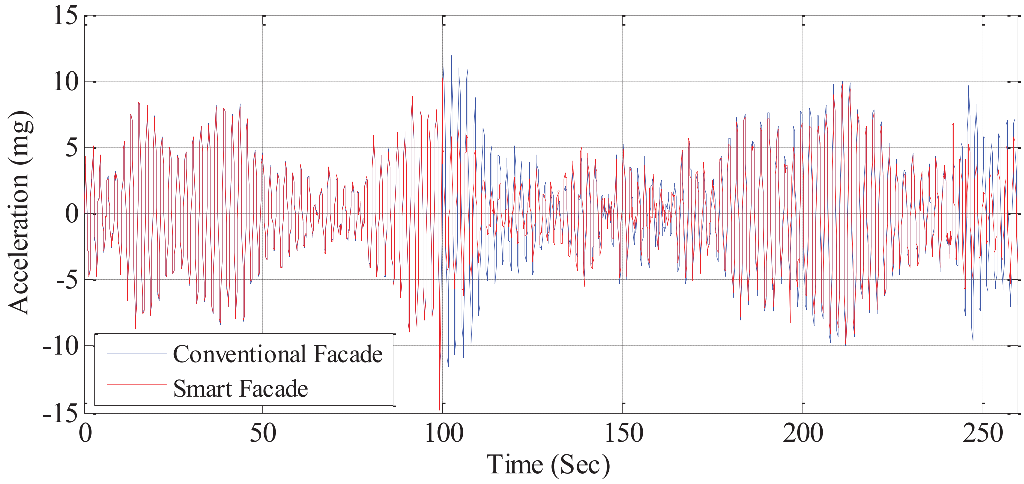


Figure A. 1 The acceleration response of conventional façade versus smart façade (Means Speed 15 m/s)

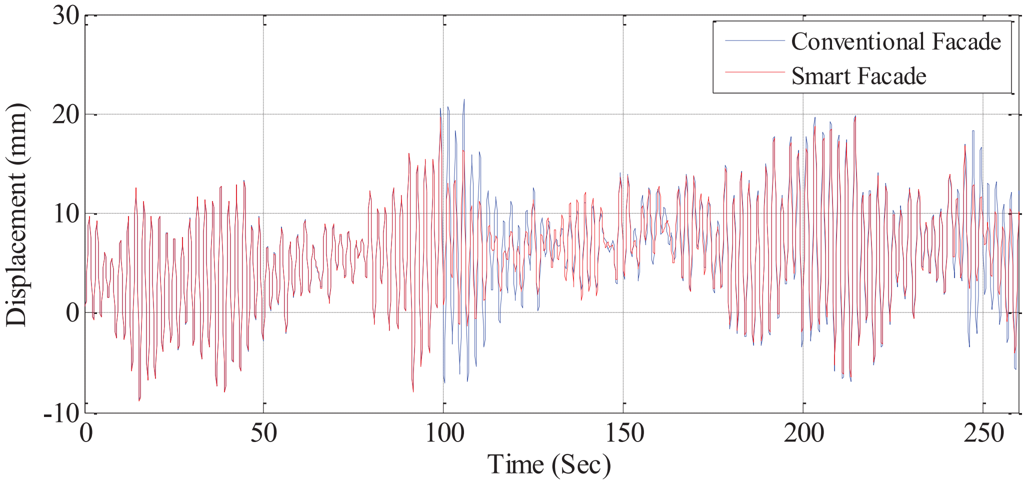


Figure A. 2 The displacement response of conventional façade versus smart façade (Means Speed 15 m/s)

Displacement response as well as acceleration has some beneficial effects (Figure A.2). The beneficial effect in displacement response started from 100sec however, in acceleration response it started a bit after 100sec. Figure A.3 shows the overview of responses comparing conventional and smart façade, in both cases there is a negligible uniform reduction.

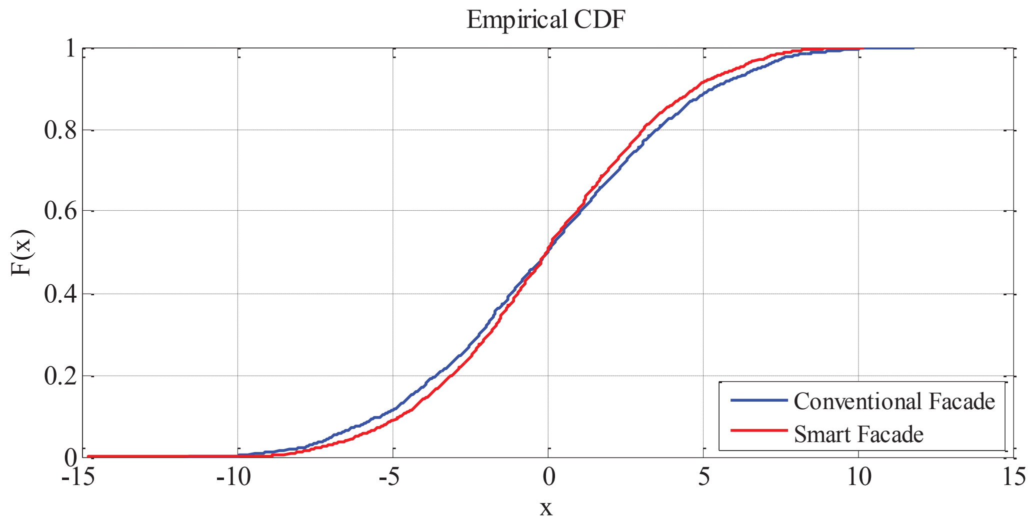


Figure A. 3 Cumulative density function of conventional vs smart façade response due to wind excitation (Means Speed 15 m/s)

The response of smart façade under 18m/s wind load shows better performance comparing to 15m/s. Figure A.4 shows the acceleration response and comparisons between conventional and smart system.

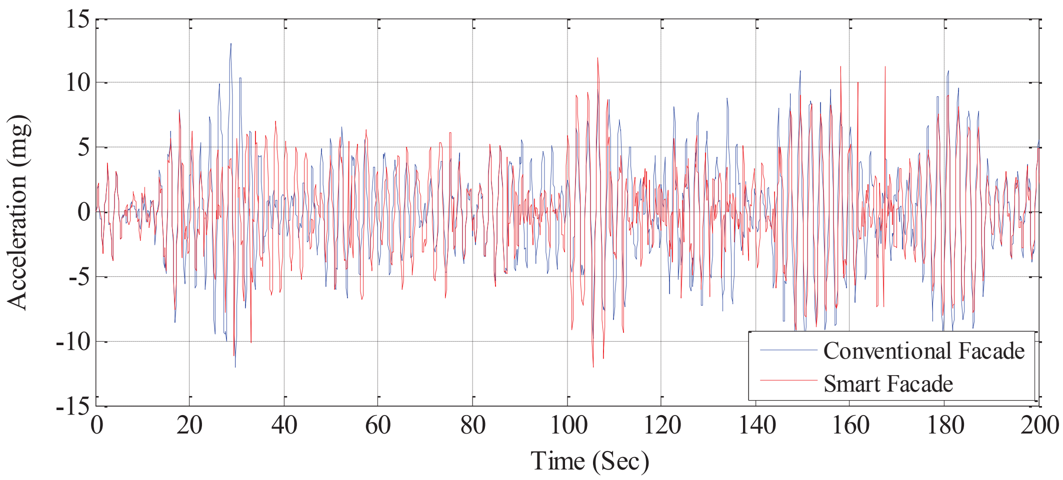


Figure A. 4 The acceleration response of conventional façade versussmart façade (Means Speed 18 m/s)

Time history of lateral displacement under 18m/s wind load are shown in figure A.5. As it was presented, acceleration response of the overall beneficial effect is negligible, although the system controls the displacement picks, in some part it makes the response worse.

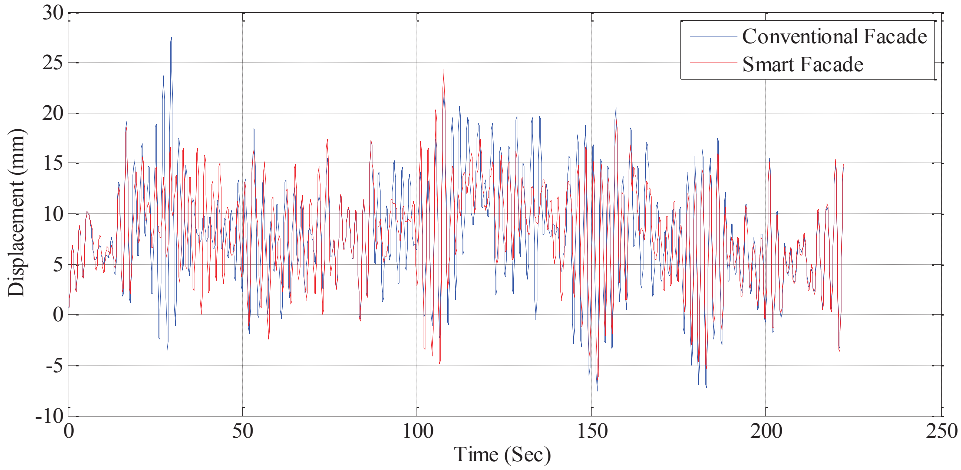


Figure A. 5 The displacement response of conventional façade versus smart façade (Means Speed 18 m/s)

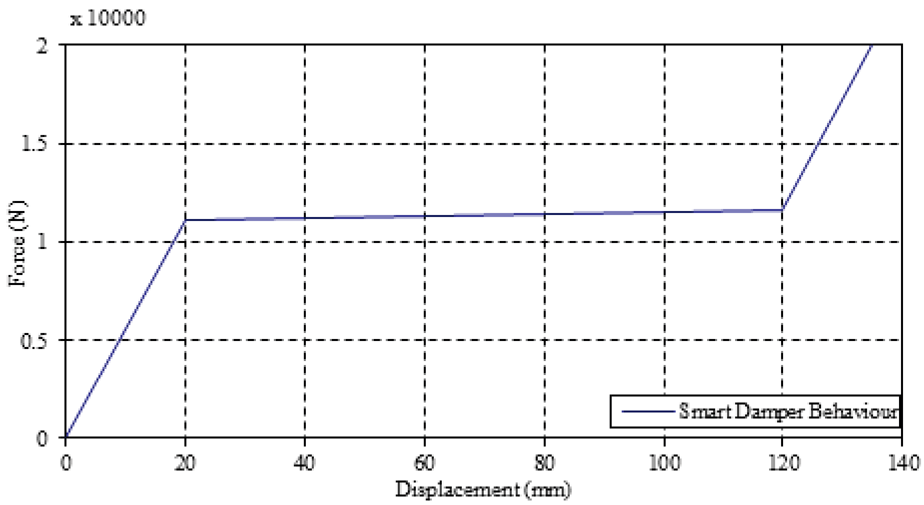


Figure A. 6 Behaviour of smart damper due to wind excitation (Means Speed 18 m/s)

Figure A.6 illustrates the performance of the damper less than 18m/s mean wind speed which leads damper to behave non-linear. For the smart damper design to behave differently regarding to input force, the 18m/s wind force could change the initial behaviour to nonlinear behaviour which leads to have some beneficial effect. Optimised design is needed to eliminate the useless movement. As shown in figure A.7 the performance of the smart system is not noticeable which means that the façade panel moves and reach to the physical limitation (140mm) of damper behaviour although it doesn't have noticeable beneficial effects.

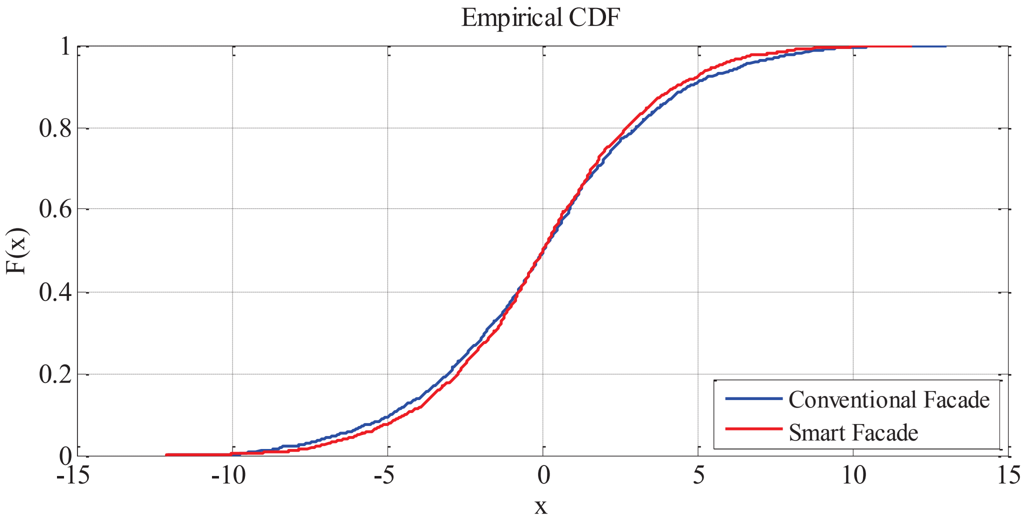


Figure A. 7 Cumulative density function of conventional versus smart façade response due to wind excitation (Means Speed 18 m/s)

Depending on the input force, the system adapts the behaviour to ensure optimum response; dampers in 27m/s mean wind speed remain mostly in the third phase as shown in Figure A.8.

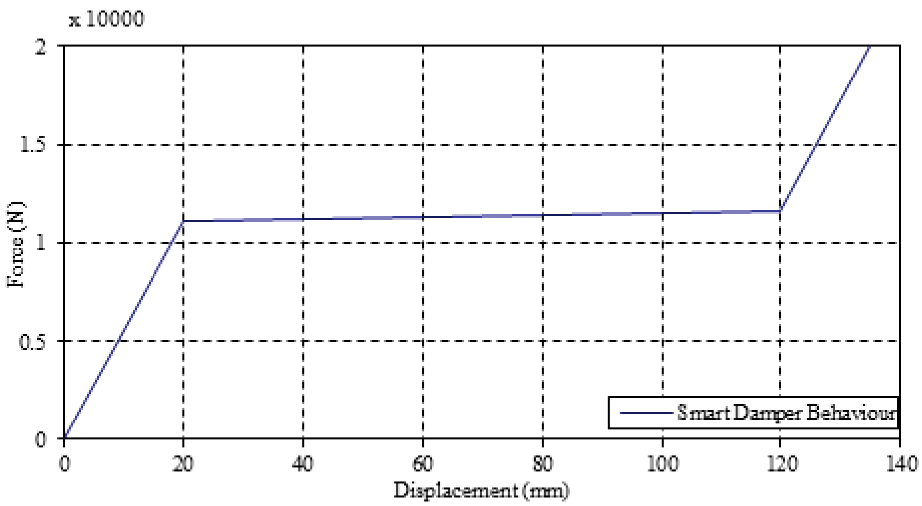


Figure A. 8 Behaviour of smart damper due to wind excitation (mean wind speed of 27 m/s)

By increasing the mean wind speed the dampers stiffness is increased which is within the third phase of the response. Remaining in phase three or the hardening phase results in reduction in panles vibration. This entire occurrence results in reducing the beneficial effects.

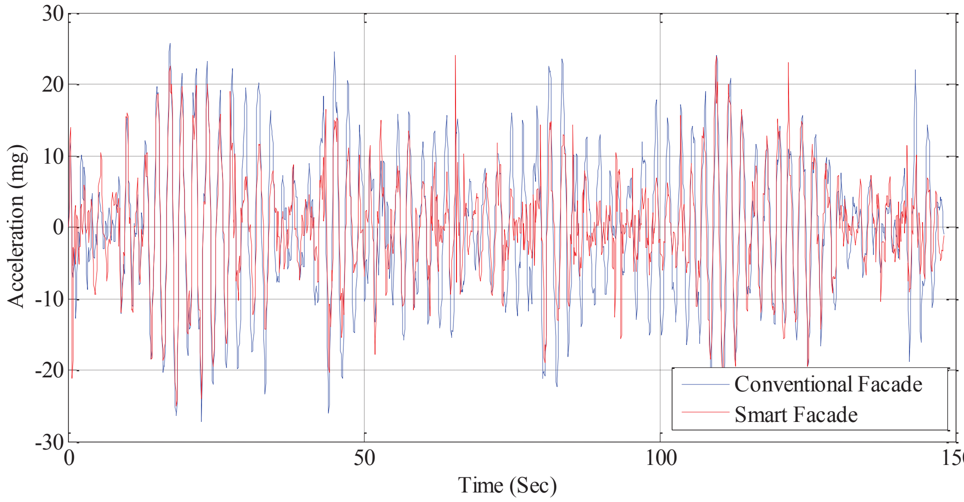


Figure A. 9 The acceleration response of conventional façade vs smart façade (mean wind speed of 27 m/s)

The curves in Figure A.10 are meant to provide a relative lateral displacement comparison of conventional and smart façade system. It can be concluded that façade system can reduce the acceleration and displacement response up to 30% due to the particular design speed of 23m/s.

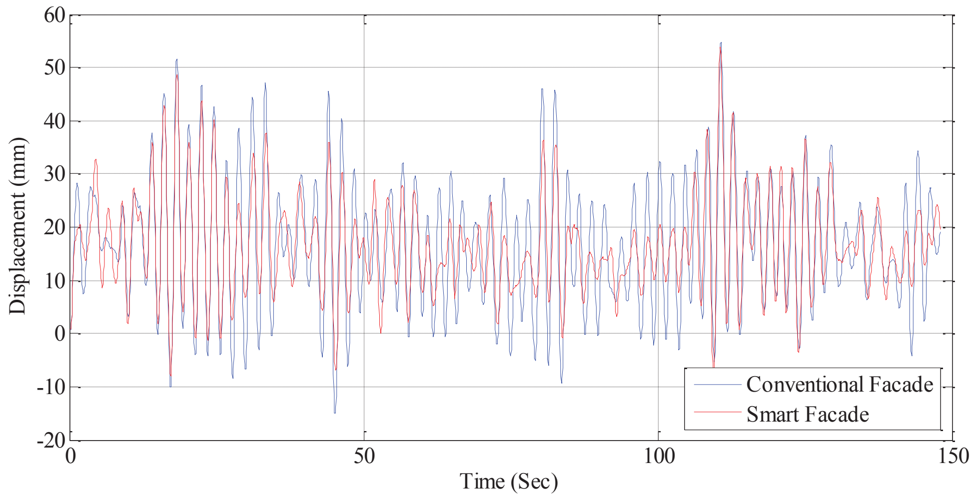


Figure A. 10 The displacement response of conventional façade vs smart façade (mean wind speed of 27 m/s)

A significant effect that requires consideration during the smart damper design is the effective wind speed. Figure A.11 shows the difference of conventional and smart façade when the wind speed is 7m/s away from design winds.

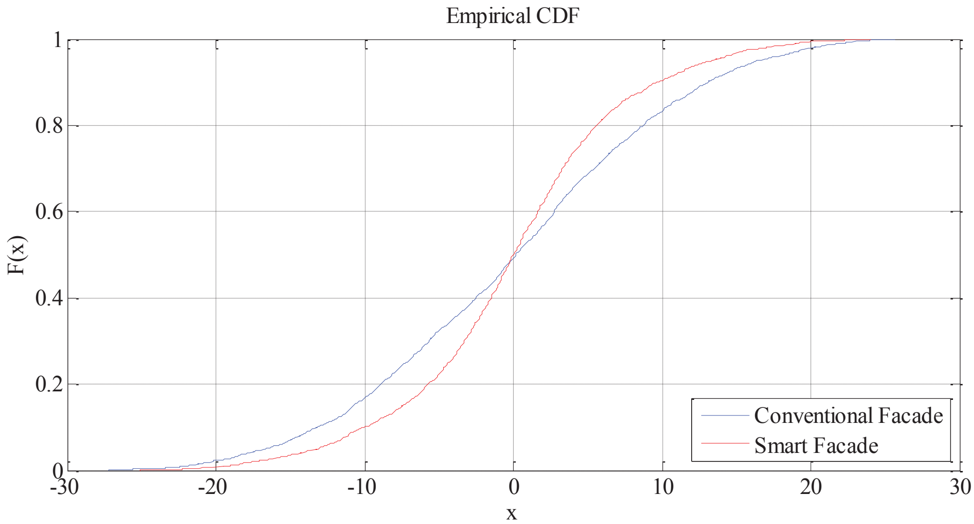


Figure A. 11 Cumulative density function of conventional vs smart façade response due to wind excitation (mean wind speed of 27 m/s)

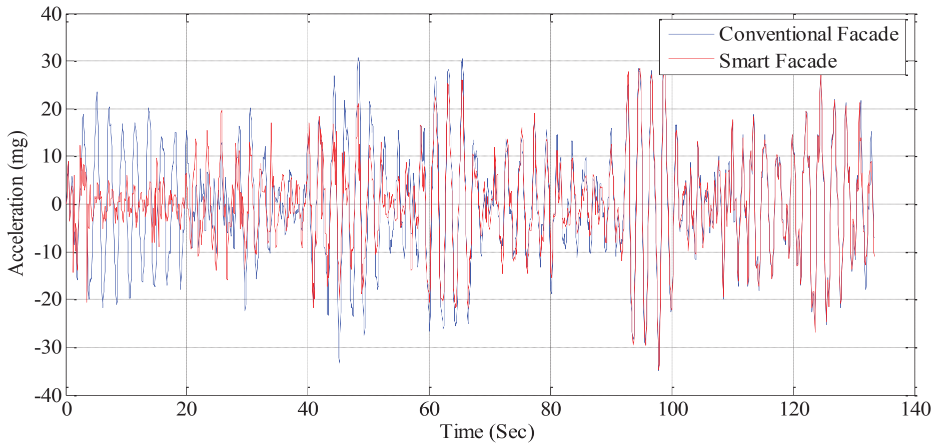


Figure A. 12 The acceleration response of conventional façade vs smart façade (mean wind speed of 30 m/s)

It can be shown that increasing the wind speed from 27m/s to 30m/s is not that noticeable where changing from 20m/s to 27m/s is quite significant. Figures A.12and A.13illustrate the acceleration and displacement response of the structure due to 30m/s wind speed, which show 20% reduction.

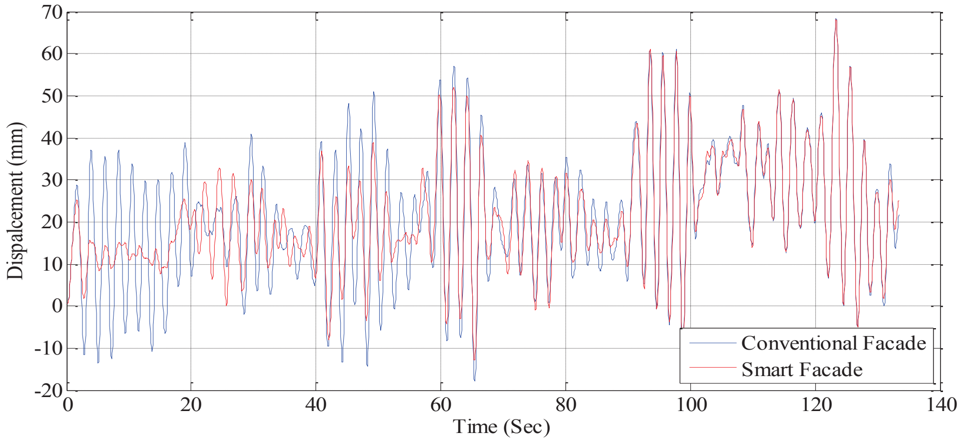


Figure A. 13 The displacement response of conventional façade vs smart façade (mean wind speed of 30 m/s)

Despite the increase in the mean wind speed, the façade system can break the panel's movement and stop the excessive motion. Figure A.14 shows that the system works between 120mm to 170mm when the wind speed is 30m/s. Due to the system configuration the hardening part (last slope) is designed to stop the excessive movement of façade panels.

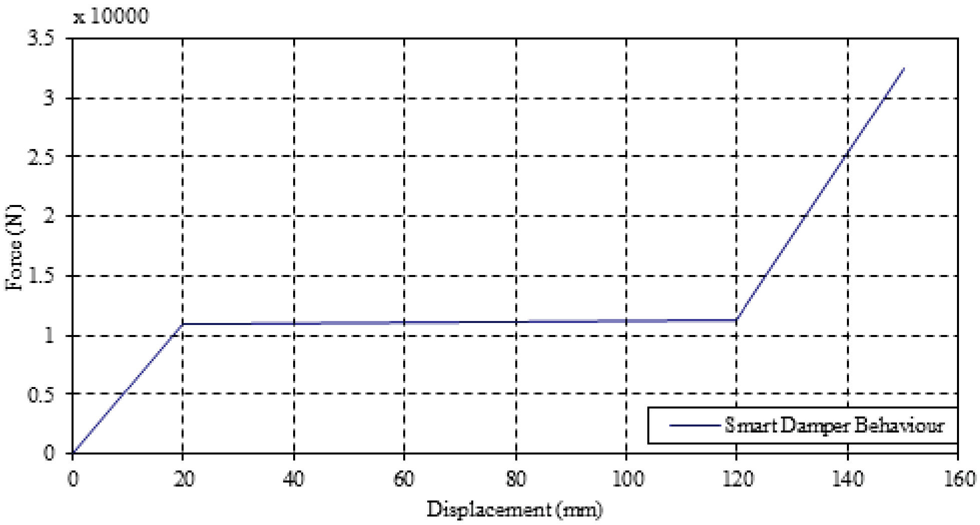


Figure A. 14 Behaviour of smart damper due to wind excitation (mean wind speed of 30 m/s)

As shown in figure A.14, 30m/s mean wind speed could decrease the performance of the system by pushing the damper behaviour to the third zone or brake zone. Cumulative density function of the two systems is presented in Figure A.15 to show the performance of the system under high wind pressure.

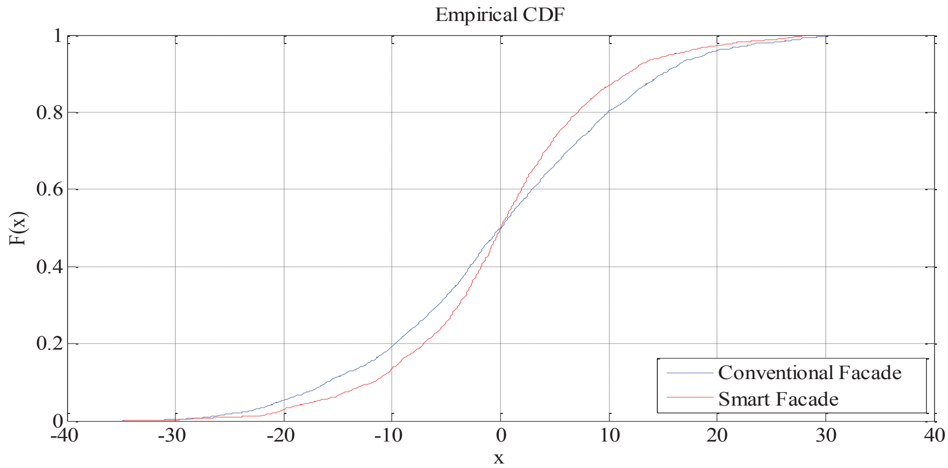


Figure A. 15 Cumulative density function of conventional vs smart façade response due to wind excitation (mean wind speed of 30 m/s)

Mid-Rise Structure 3D Analysis

Figure A.16 presents the time history comparison analysis of the conventional structure and smart damper façade system. In this case due to low mean wind speed, the smart façade damper does not activate and behaves linearly.

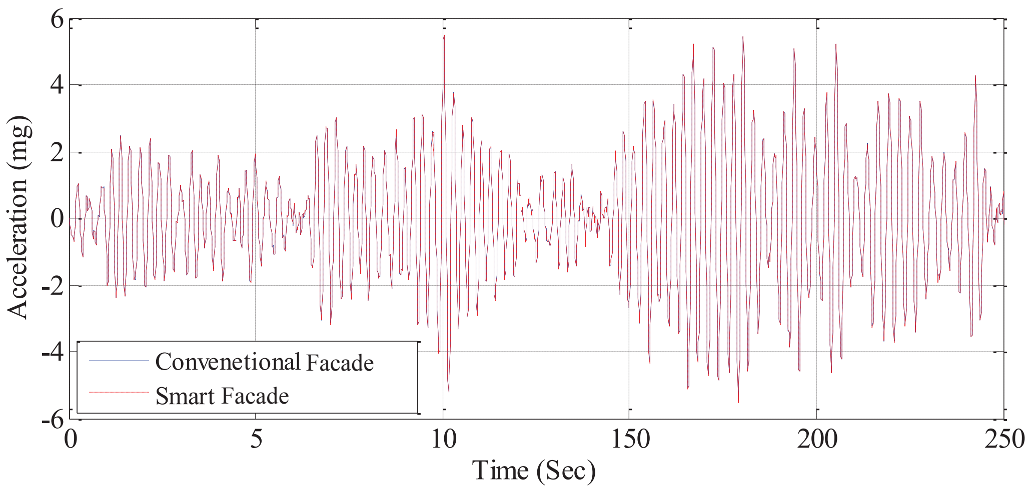


Figure A. 16 Acceleration response of structure with and without smart façade system subjected to 15m/s mean wind

The system is designed in a way that does not get activated by the daily wind speed. The displacement is intended to stay below 20mm which is the phase one of the actual behaviour of the damper façade system (FigureA.17).

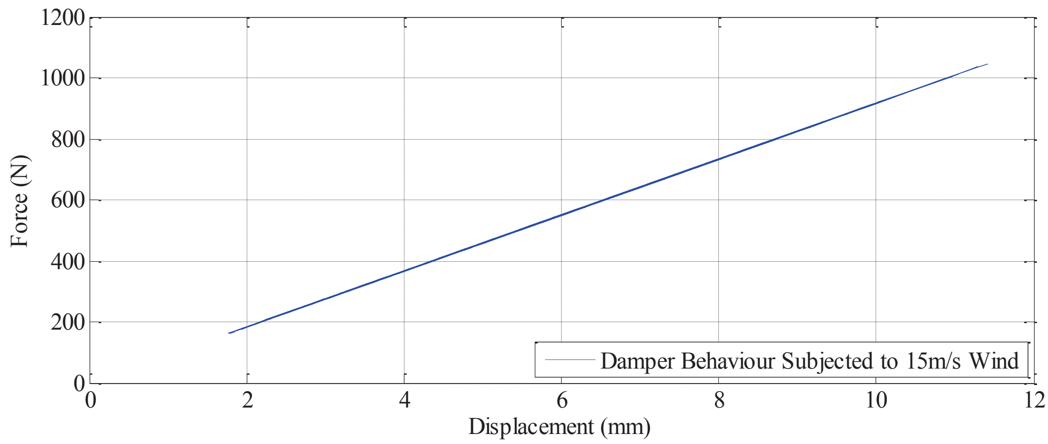


Figure A. 17 Behaviour of smart damper due to wind excitation (mean wind speed of 15 m/s)

Upon increasing the mean wind speed from 15m/s to 20m/s the system enters phase two behaviour causing the panels to vibrate in a certain manner (Figure A.18). This vibration and the change from phase one to phase two activates the damper system. When the wind force increases, the system changes from low stiffness to high stiffness which then causes the façade panels to reach a certain frequency. In line with the above discussion, as shown in Figure A.19, the acceleration response of the structure decreases.

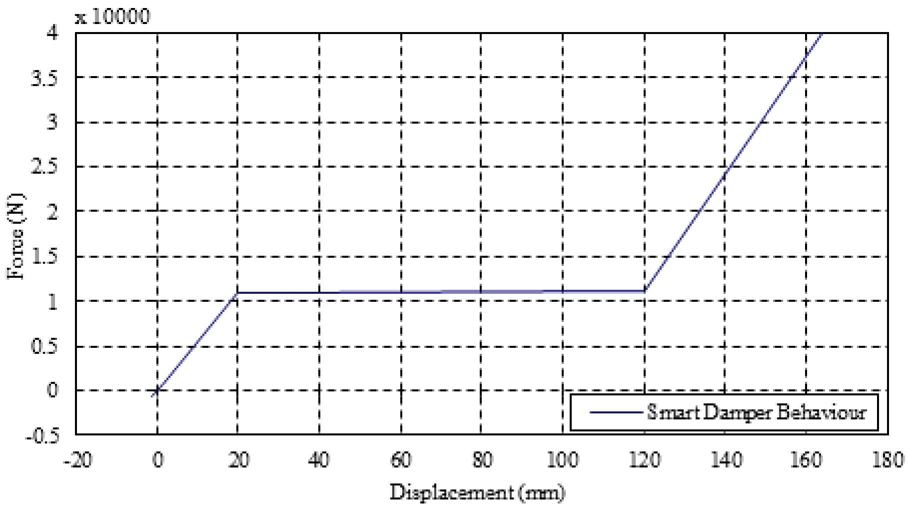


Figure A. 18Behaviour of smart damper due to wind excitation (mean wind speed of 20 m/s)

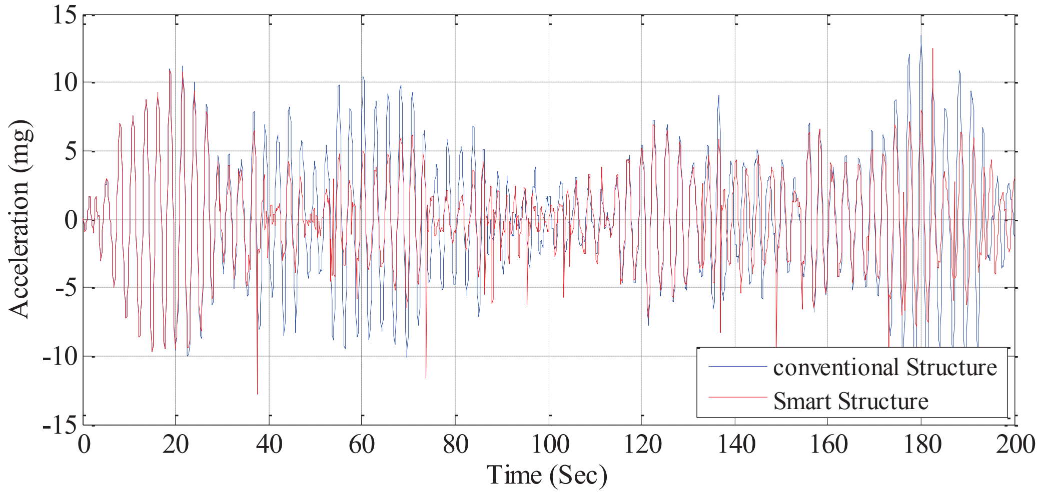


Figure A. 19 Acceleration response of structure with and without smart damper façade system subjected to 20m/s mean wind speed

Once the wind speed increases by 2m/s and reach mean speed of 25m/s, damper system works in the third phase. In this phase the stiffness of the damper system increases. therefore, the vibration is lowered. Figure A.20 illustrates the above explanation.

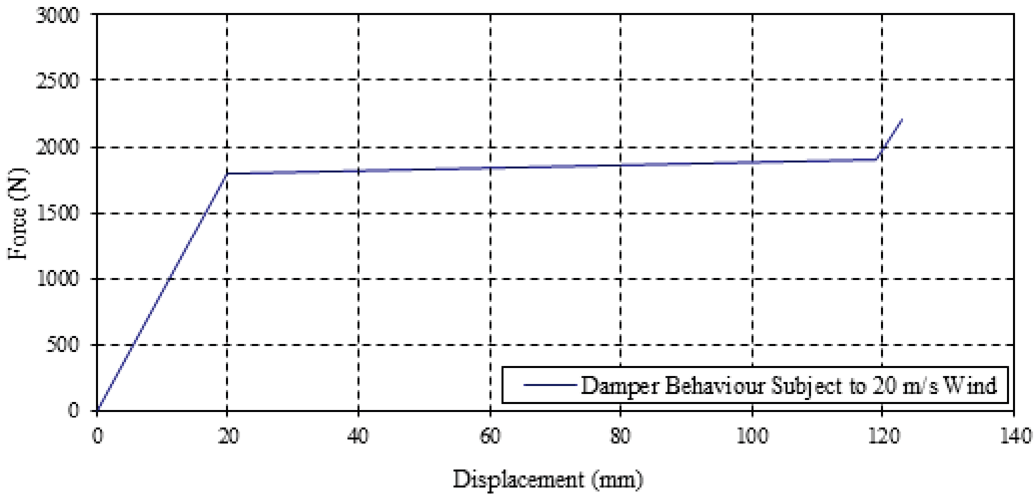


Figure A. 20 Behaviour of smart damper due to wind excitation (Means Speed 25 m/s)

Based on the above argument, the effectiveness ratio of the system drops when the wind speed increases from the designed mean wind speeds to 25m/s. The mitigation of the response of the structure changes from 50% to 20% when the mean wind speed changes from 23m/s to 25m/s due to system high sensitivity (Figure A.21).

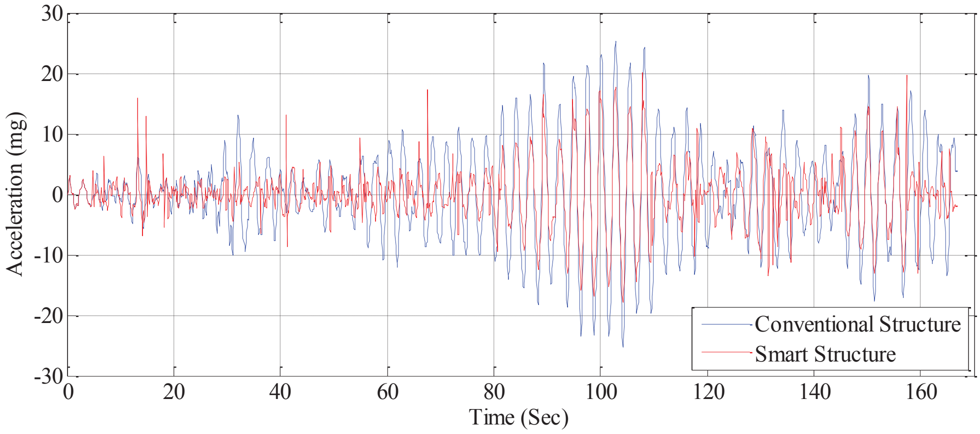


Figure A. 21 Acceleration response of structures with and without smart damper façade system subjected to 25m/s mean wind

In summary, the sensitivity of the smart façade damper system subjected to variable mean wind speeds has been investigated in the 2D and the 3D structure. In either case the smart system reduces the response of the main structure and proves effective. The 3D model which in fact corresponds to real situations indicates that the system is sensitive to any slight changes in mean wind speed.

High-Rise Structure 2D Analysis

Figure A.22 presents the response of the structure subjected to 12 m/s wind speed. Based on the damper façade design it is expected that the main structure behave similar to the structure equipped with the smart façade system.

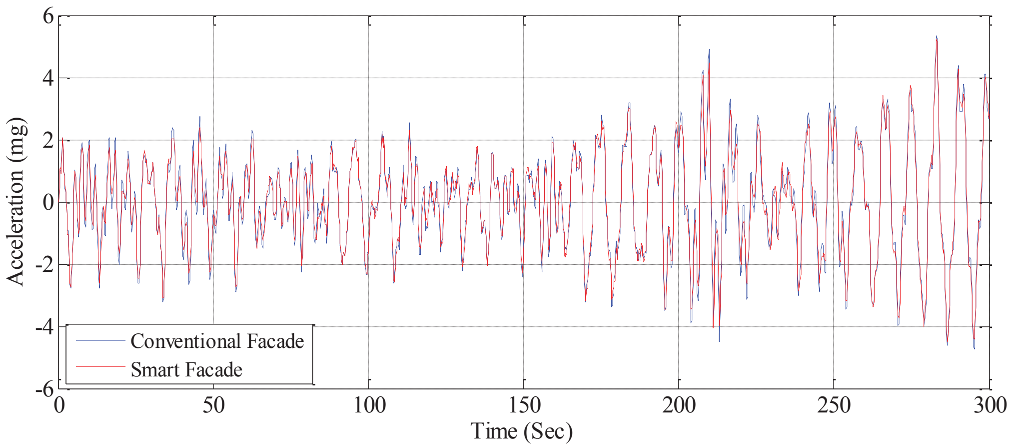


Figure A. 22 The acceleration response of conventional façade versus smart façade (mean wind speed of 12m/s)

Daily winds in the movable part of façade could be harmful in terms of the comfort level of people and also it is important to ensure that façade system does not activate below certain wind speeds. It is important to also demonstrate that the damper façade system could not make the response of the main structure worse than conventional structures. As shown in Figure A.23, the two systems work similarly when subjected to 12 m/s wind speed. Wind speed of 12 m/s leads to activating the first phase of the damper system which is linear and can control the panels to move less than 20mm. Figure A.24 shows the trend of panels movement subjected to the wind loads.

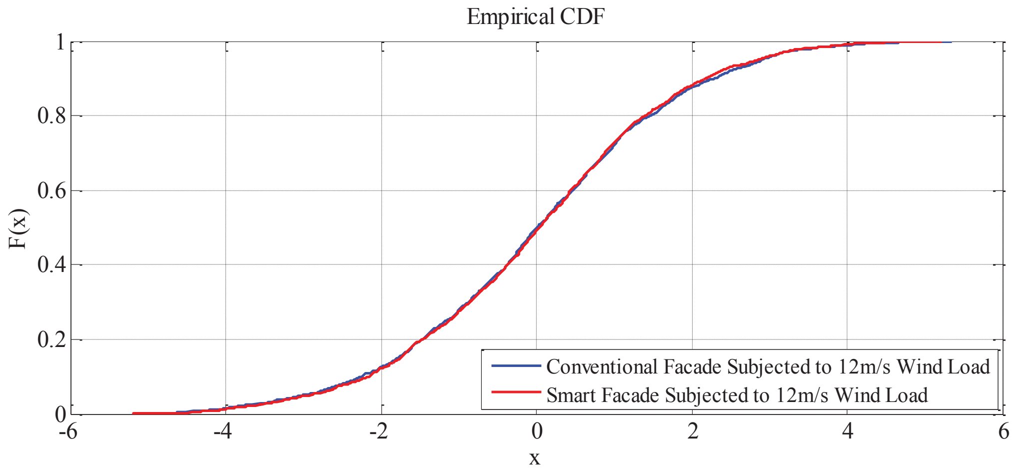


Figure A. 23 Cumulative density function of conventional versus smart façade response due to wind excitation (mean wind speed of 12m/s)

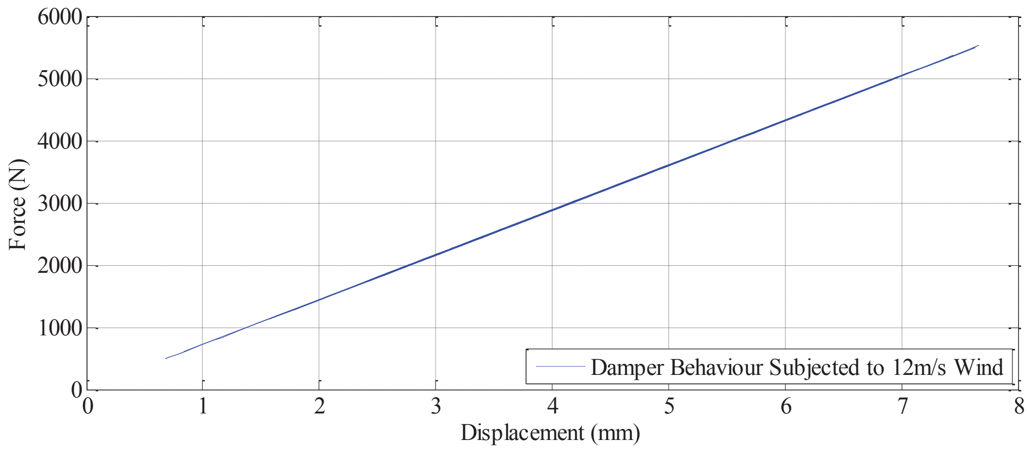


Figure A. 24 Behaviour of smart damper due to wind excitation (mean wind speed of 12m/s)

Increasing the wind speed leads to greater movements in the damper façade, although it can be seen in Figure A.25 that smart damper subjected to 15 m/s mean wind speed reaches up to 15mm of displacement, it still remains in the first phase of behaviour. The displacement differences of façade dampers under two wind speeds are noticeable by comparing Figure A.24 to Figure A.25. Changing the mean wind speed from 12 m/s to 15 m/s leads to the displacement of the damper to increase from 7.5 mm to 14.9 mm.

Being in the first phase, in this range of wind speeds it is expected that the structure equipped with smart devices will behave similar to the conventional structures.

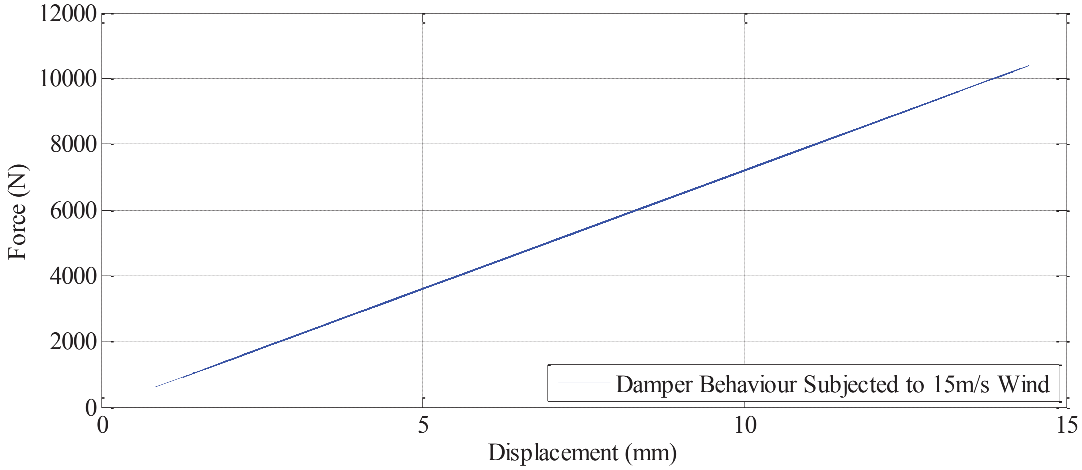


Figure A. 25 Behaviour of smart damper due to wind excitation (mean wind speed of 15m/s)

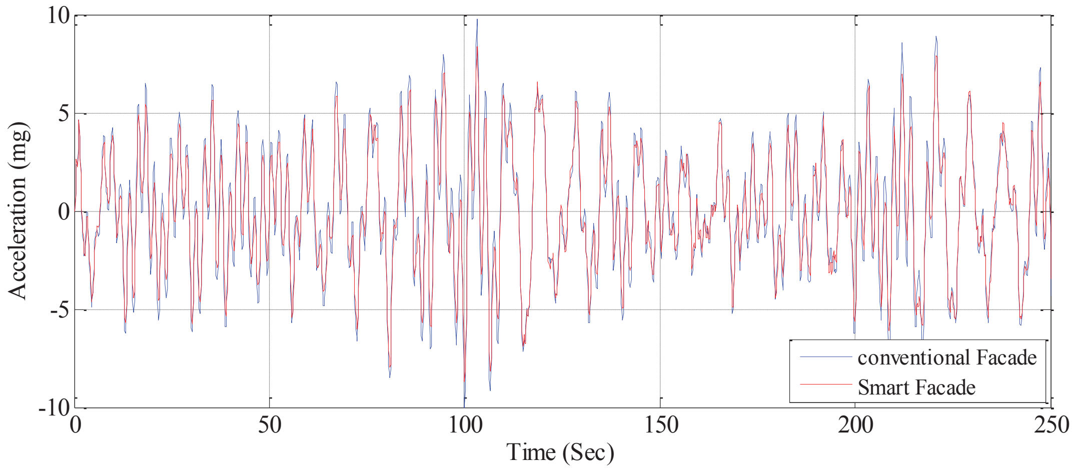


Figure A. 26 The acceleration response of of conventional vs smart façade response due to wind excitation (mean wind speed of 15m/s)

It can be concluded from both cumulative density function trend (Figure A.27) and time history response (Figure A.26) that, the response of conventional façade system and new movable façade concept subjected to 15m/s mean wind speed is quite similar.

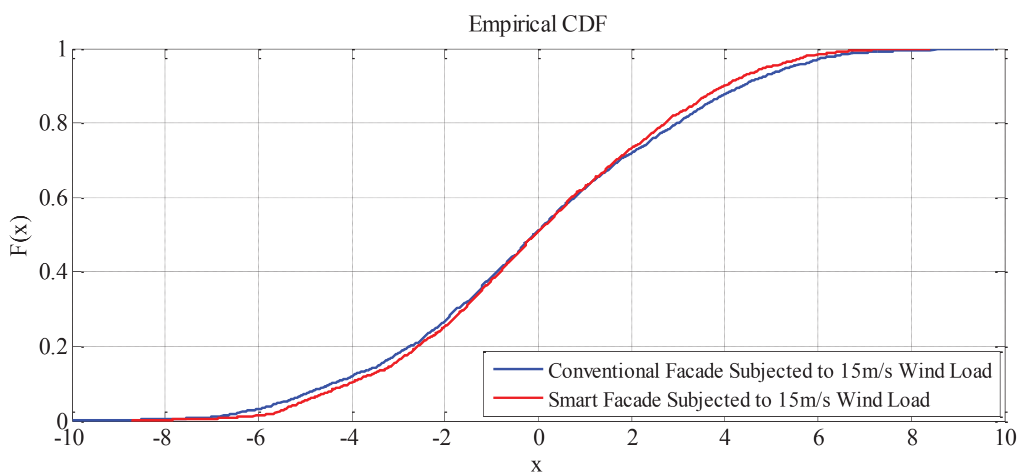


Figure A. 27 Cumulative Density Function of Conventional vs Smart Façade Response due to wind excitation (mean wind speed of 15m/s)

Based on Figure A.28, it can be noted that when the wind speed is increased to 18 m/s the damper will reach the change of phase point. It is worth noting that with 18 m/s wind speed the panels remain fixed or with a small relative displacement; therefore, as expected the behaviour of the damper façade system should be similar to the conventional system as shown in Figures A.29 and A.30.

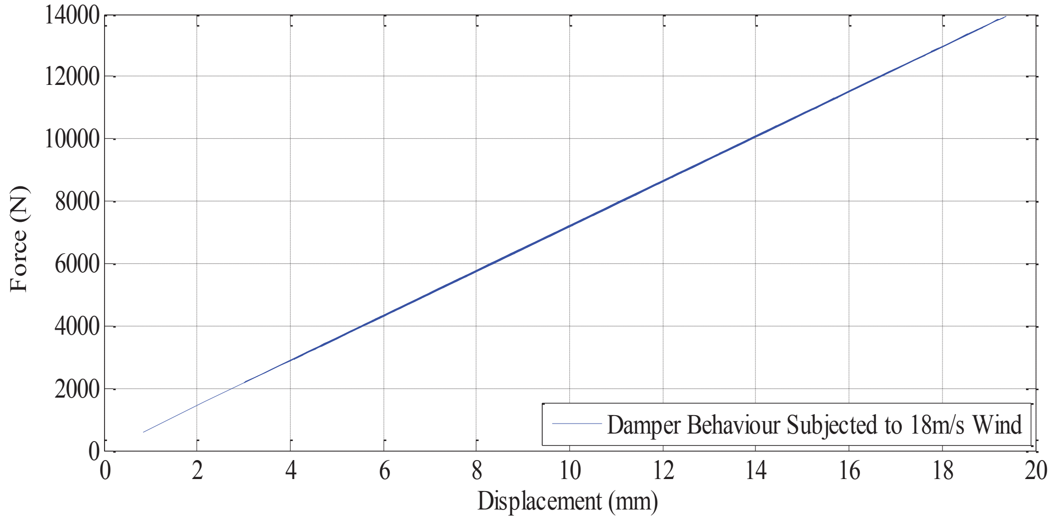


Figure A. 28 Behaviour of smart damper due to wind excitation (mean wind speed of 18 m/s)

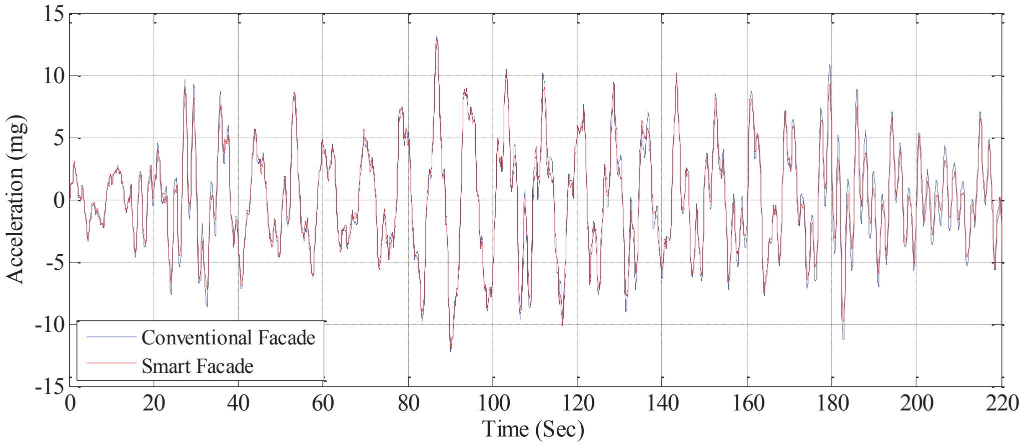


Figure A. 29 The acceleration response of conventional façade versus smart Façade (mean wind speed of 18 m/s)

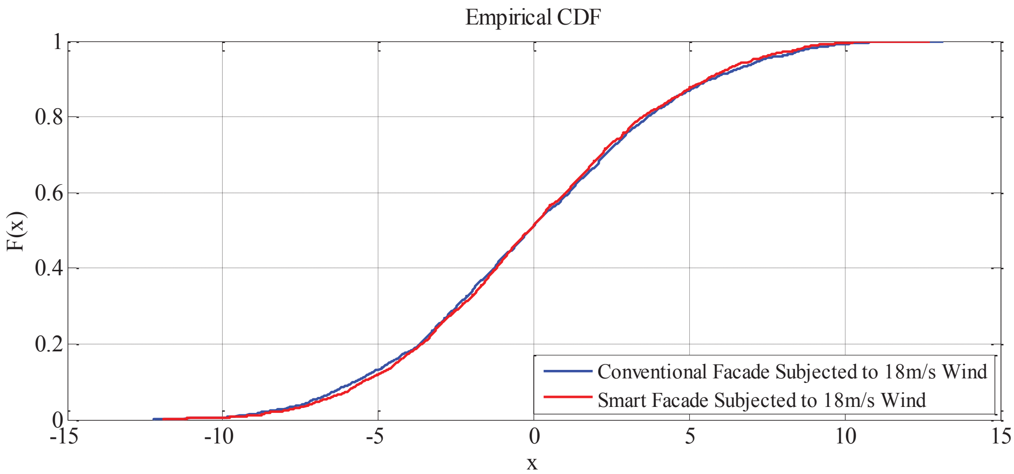


Figure A. 30 Cumulative density function of conventional vs smart façade response due to wind excitation (mean wind speed of 18 m/s)

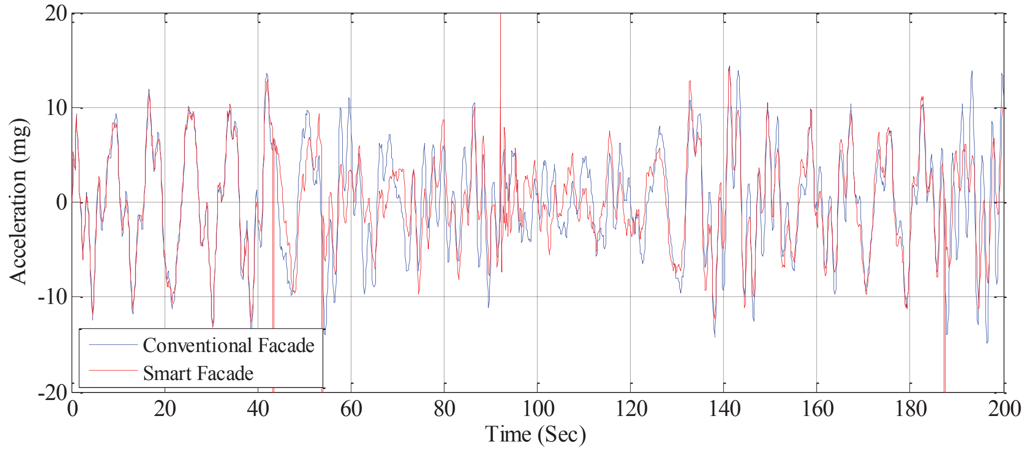


Figure A. 31 The acceleration response of conventional façade vs smart façade (mean wind speed of 20 m/s)

As stated earlier the transfer point of the damper concept is designed to occur when the mean wind speed reaches 20 m/s. As shown in Figure A.32, this mean wind speed leads to change of behaviour of the smart damper from the first phase (high stiffness ratio phase) to the second phase (low stiffness ratio phase).

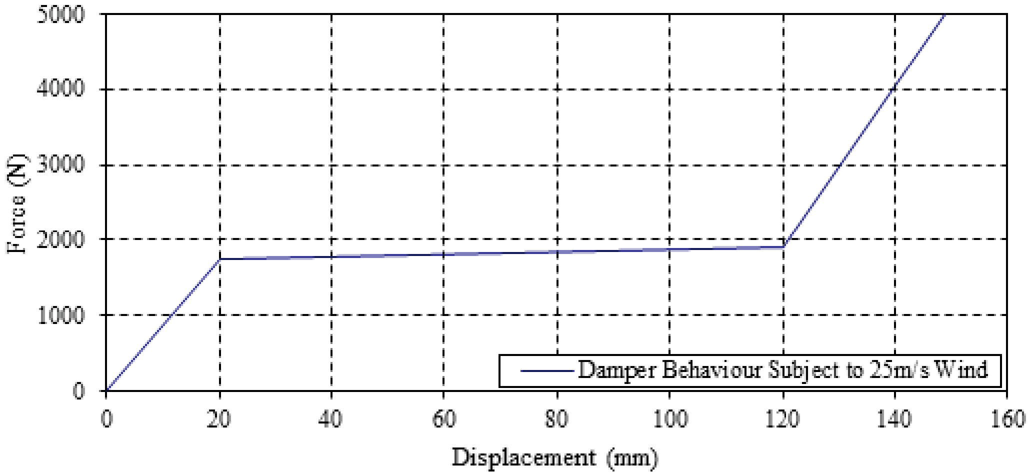


Figure A. 32 Behaviour of smart damper due to wind excitation (mean wind speed of 20 m/s)

The second slope is designed to let the panels vibrate at a certain frequency which leads to reducing the acceleration and displacement response of the main structure. Based on the result obtained from Figure A.31 and A.33, the smart system is starting to work, but the panel vibration is not yet enough to make 50% reduction in response.

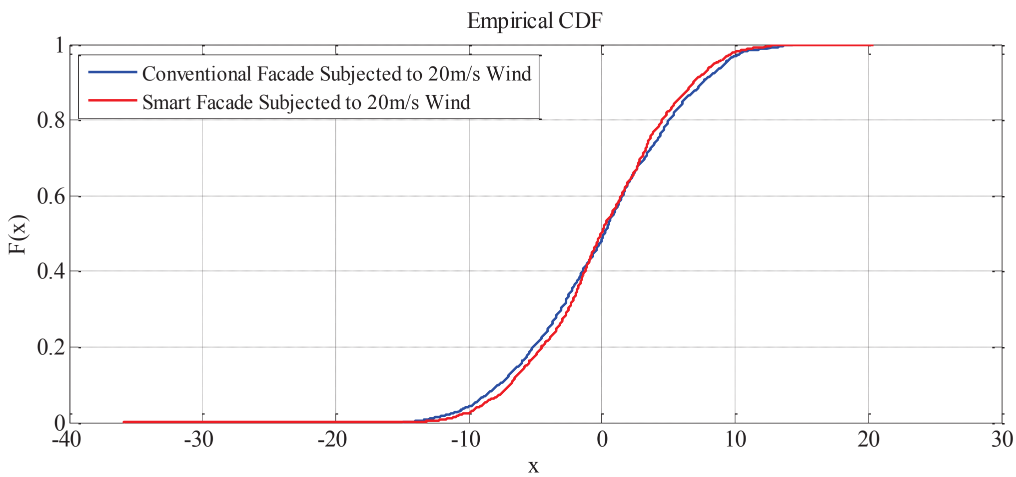


Figure A. 33 Cumulative density function of conventional vs smart façade response due to wind excitation (mean wind speed of 20 m/s)

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