
Enhancement of Structural Analysis of Multi-storey Buildings by Integrating Non-structural Components into Structural System

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ABSTRACT

In the last few decades there has been an enormous increase in the number of high-rise buildings worldwide. Current Australian high-rise building design practice is to assume that the structural skeleton of a building provides resistance to any lateral forces that might occur. The overall design of high-rise buildings is usually dominated by serviceability limit state considerations rather than the ultimate limit state factors.

Various structural forms and materials have been developed and adopted in the construction of high-rise buildings. The structural response depends on the structural form and materials utilised and also on the interaction between structural components and non-structural components for that buildings are widely recognised as a complex assemblage of both structural skeleton and non-structural components (Su *et al.* 2005). The lateral performance of high-rise buildings is complex because of the conflicting requirements of diverse (structural and non-structural) building systems (Hutchinson *et al.* 2006). There is a scope to improve the serviceability limit state design requirements over the traditional approach.

The aim of this study is to analyse the structural performance based on evaluations of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis. To achieve this specific aim, buildings in various locations were investigated; finite element analyses on a case-study building were carried out, followed by the laboratory testing as the validation of parameters and a parametric study to evaluate the influence of integrating non-structural components into the structural analysis on the overall building performance.

It is discovered that by integrating non-structural components into the structural analysis, building performance differs significantly. From the analyses in this study, when including different non-structural components in the structural analysis, the total stiffness of the building is significantly increased, to more than 50%, depending on the key influencing factors, which are discussed in this study: quantity, location, and connection properties assigned to the non-structural components.

It is also noticed that the natural frequencies of the structure change when different non-structural components are included in the analysis.

In terms of the stress distribution, by including non-structural components in the structural analysis, the bending moment and shear force distributed in the structural components, such as columns, change accordingly. These changes are related to their relevant locations to the specific non-structural components.

The damage level of different non-structural components was also assessed. The maximum allowable structural movements defined by the Australian Standard were applied to the individual non-structural component. It is concluded that if not being delicately isolated from the primary structure, the precast concrete infill panels will not be able to accommodate the amount of stress transferred from the primary structure.

Based on the results obtained from this study, it is concluded that integrating non-structural components into the structural analysis has significant influence on the serviceability of the overall structural system. Damages to the non-structural components caused by the interactions between the primary structure and the non-structural components are also remarkable, even if the whole building system is under service loads. Consequently, the structural analysis method adopted by the current design practice is suggested to be updated.

DECLARATION

This is to certify that

- (i) the thesis comprises only my original work towards the PhD except where indicated in the Preface,
- (ii) due acknowledgement has been made in the text to all other material used,
- (iii) the thesis is less than 100,000 words in length, exclusive of tables, maps, bibliographies and appendices.

Bing Li

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PREFACE

In the course of this study, 3 peer reviewed journal papers, 5 peer reviewed conference papers and 1 research report have been produced.

Chapter 2 has in part, been published and presented in the following publications:

- Hutchinson, G. L., Collier, P., Duffield, C. F., Gad, F. E., Li, B. and Mendis, P. A. (2006). “Integration of GPS Measurement and Secondary Structural Elements into the Serviceability Design of Structures.” *Proceedings of EASEC-10*, Bangkok, Thailand.

Chapter 4 and 5 has in part, been published and presented in the following publications:

- Li, B., Duffield, C. F. and Hutchinson, G. L. (2007). “Field Investigation Report - Analysis of Connection Properties.” *Department of Civil & Environmental Engineering Research Report*, RR/Struc/11/2006, The University of Melbourne.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2007). “The Influence of Structural and Non-structural Components on the Lateral Performance of High-rise Buildings.” *Proceedings of IESC-4*, Melbourne, Australia.

Chapter 6 has in part, been published and presented in the following publications:

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Chapter 7 has in part, been published and presented in the following publications:

- Li, B., Hutchinson, G. L. and Duffield, C. F. (2009). “The Influence of Non-structural Components on Tall Building Stiffness.” *The Structural Design of Tall and Special Buildings*, DOI: 10.1002/tal.565, In Press.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2009). “The Influence of Non-structural Components on the Serviceability Performance of High-rise Buildings.” *Australian Journal of Structural Engineering*, Vol.10, No. 1, pp. 53-62.

- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “Quantification of the Contribution of Non-Structural Components to the Performance of High-rise Buildings.” *Proceedings of EASEC-11*, Taipei, Taiwan.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “A Parametric Study of the Lateral Performance of a High-rise Structure.” *Proceedings of ASEC 2008*, Melbourne, Australia.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “Simplified Finite Element Modelling of Multi-storey Buildings: The Use of Equivalent Cubes.” *Electronic Journal of Structural Engineering*, Vol.8, pp. 40-45.

These publications are contained in Appendix I.

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NOMENCLATURE

a	ratio of the design ground acceleration a_g to the acceleration of gravity g
a_c	component amplification factor 2.5 for flexible spring-type mounting systems for mechanical equipment (unless detailed in dynamic analysis is used to justify lower values) 1.0 for all other mounting systems
a_e	long end distance
a_{floor}	effective floor acceleration at the level where the component is situated
a_p	component amplification factor
a_s	tensile stress area
A	the sum of the areas of right and left walls
A_c	sectional area of the column
A_d	the cross section area of the brace
C	$C = \sum (I_c)_i / h_i$, for the columns in storey i of the bent; $\sum (I_c)_i$ is the sum of the second moment of area of the columns in one storey h_i is the storey height
d_f	bolt diameter, has been chosen as 20 mm
d_r	design inter-storey drift
D_p	relative seismic displacement
D_r	drift ratio
E	Young's Modulus of the material
EI_0	bending stiffness of the outrigger
f_{pu}	nominal bearing strength of the ply
f_{uf}	tensile strength of the bolt
f_{vf}	average shear strength of the bolt
F_a	horizontal seismic force, acting at the centre mass of the non-structural element in the most unfavourable direction

F_c	forces generated on the part or component
F_p	component design force applied horizontally at the centre of gravity of the component or distributed according to the mass distribution of the component
F_{pv}	component seismic design force applied vertically at the centre of gravity of the component or distributed according to the mass distribution of the component
G	$G = \sum (I_g)_i / L_i$, for the girders across one floor level i of a bent $\sum (I_g)_i$ is the sum of the second moment of area of the beams in storey i L_i is the distance between columns at storey i
GA	the shearing rigidity
H	the total height of the structure
h	average roof elevation of structure, relative to grade elevation
h_i	storey height at level i
$h_{i-1/2}$	the height from bottom of the structure to the middle of storey i
I	the sum of the second moments of area of individual left and right side walls
I_b	the second moment of area of the coupling beam
I'_c	component importance factor, taken as: 1.5 for components critical to life safety 1.0 for all other components
I_c & I_g	the second moment of area of the column and girder respectively
I_p	component performance factor; 1.0 shall be used for the Life Safety Non-structural Performance Level and 1.5 shall be used for the Immediate Occupancy Non-structural Performance Level
I_w	the second moment of area of the wall
l	the distance between the central lines of two walls
l_s	the test span for the four-point bending test
M_0	the moment at the base of the structure
M_c	the restraining moment on the wall due to axial forces in the columns of the façade structure

$M_{i-1/2}$	the moment at the mid-height of storey i
N_{tf}	the tensile capacity of a bolt
q_a	behaviour factor of the element
Q_i	lateral force on level i
r_n	distance from centre of rotation to the applied force
R_c	component ductility factor 1.0 for rigid components with non-ductile or brittle materials or connections 2.5 for all other components and parts
R_p	component response modification factor
S	the flexibility parameter for the vertical structure
S_a	seismic coefficient pertinent to non-structural elements
S_c	$= (P/\delta)_c$, is the slope of the straight line portion of the load-deflection graph from the four-point bending test
S_b	$= (P/\delta)_b$, is the slope of the straight line portion of the load-deflection graph from the three-point bending test
S_l	the flexibility parameter for the horizontal structure (outrigger system)
S_r	the flexibility parameter for the horizontal structure (multi-bay façade system)
S_{XS}	spectral response acceleration parameter at short periods for any earthquake hazard level and any damping determined in the standard
t_p	thickness of the ply
T_l	fundamental vibration period of the building in the relevant direction
T_a	fundamental vibration period of the non-structural element
U	height of upper support attachment at level x as measured from grade
u	height of lower support attachment at level y as measured from grade
v	reduction factor to take into account the lower return period of the seismic event associated with the serviceability limit state
v_n^*	design force per unit length of weld normal to the plane of the fillet weld throat

v_l^*	design shear force per unit length of weld longitudinal to the plane of the fillet weld throat
v_t^*	design shear force per unit length of weld transverse to the plane of the fillet weld throat
V^*	design action
V_b	nominal bearing capacity of a ply
V_{bc}	the force on each bolt under the design action couple V_e^*
V_{bu}	the ultimate bearing capacity of a ply
V_{bv}^*	the force on each bolt when design action act at the bolt group centroid
V_f	nominal shear capacity of a single bolt
V_{fn}	nominal shear capacity of a single bolt for threads intercepting one shear plane
V_{fx}	nominal shear capacity of a single bolt for a plain shank intercepting one shear plane
V_{of}^*	force on the bolt furthest
V_n^*	force on any bolt
w	the uniformly distributed lateral load
W_a	weight of the element
W_c	seismic weight of a component
W_p	component operating weight
x	the distance measured from the top
X	elevation in structure of component relative to grade elevation
(x_e, y_e)	centre of the rotation
y_{max}	the lateral deflection at the top of the wall
Z	height of the non-structural element above the base of the building
α	the structure parameter
Δ	the top deflection of the structure
δ_c	the drift caused by column flexure in storey i
δ_g	the drift caused by girder flexure in storey i

δ_i	the total drift at storey i
δ_{if}	the overall drift caused by bending in storey i
δ_s	the shear deflection of braced bents at storey i
δ_{xA}	deflection at building level x of building A, determined by analysis
δ_{xB}	deflection at building level x of building B, determined by analysis, or equal to 0.03 times the height U of level x above grade or as determined using other approved approximate procedures
δ_{yA}	deflection at building level y of building A, determined by analysis
l	distance between the columns
λ	the cross sectional shape factor for shear which equals 1.2 in the case of rectangular sections
θ_{if}	the inclination of storey i
γ_a	importance factor of the element

CHAPTER 1

INTRODUCTION

1.1 Background

High rise buildings are in high demand due to the world population boom and technology development of recent decades (Stafford Smith and Coull 1991). The lateral performance of multi-storey buildings under different loading conditions is greatly influenced by various parameters such as structural stiffness and base to height ratio of the building. Optimization and refinement of such performance has become the focus as well as the constraint for structural engineers in their design practice.

In Australia, the design process for typical 30 to 50 storey office and residential buildings involves designing a skeleton to resist the ultimate limit state and serviceability limit state loads whereas the non-structural components, such as façades and infill walls, are designed to be detached from the primary structure. However, recent research (Melchers 1990; Arnold 1991; Hall 1995; Phan and Taylor 1996; Naeim 1999; McDonnell 2001) has indicated that the structural role of “non-structural components”, in resisting lateral loads can be very significant. Interactions between non-structural components and the structural skeleton may lead to distress, loss of serviceability and occasional failure of the non-structural components. In addition, the actual performance of real buildings differs significantly from that of idealised structural models (Gad et al. 1998; 1999a; 1999b; 2000; Naeim 1999; Sugiyama 2000). Studies carried out by previous researchers (Gad et al. 1998; 1999a; 1999b; 2000; Su et al. 2005) have clearly shown that non-structural components in low-rise buildings can increase the lateral stiffness and the strength of the building by more than 100%. This accounts for the difference between the theoretical estimates and real performance.

Recognising the reality that lateral loads are carried by a combination of the skeletal structure and non-structural components gives rise to the potential for the development of a new design philosophy for serviceability loading and for modified design procedures, built on an improved fundamental understanding of how buildings actually perform.

1.2 Aim and Objectives

The ultimate aim of this project is to develop a state-of-the-art design philosophy to predict and utilise the structural performance of the overall building system. However, considering that this study is only the start of this project, the aim of this study is thus the first step towards the ultimate aim, that is, to analyse the structural performance based on evaluations of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis. To achieve this aim, the following five research objectives have to be met:

- (1) Propose and evaluate an integrated analytical system to obtain reliable data from building movements and to analyse the data;
- (2) Identify the effects of integrating non-structural components into the structural analysis on the overall building performance;
- (3) Identify the influence of connection properties to the overall structural performance;
- (4) Identify the damage level of individual non-structural components when integrating non-structural components into the structural analysis;
- (5) Propose integrated design suggestions for structural and non-structural components of multi-storey buildings.

Approaching the aim and objectives listed above is of great importance in achieving a solid knowledge base of the real global behaviour of multi-storey structures, thus leading to understanding and development of the current design industry both within Australia and abroad.

1.3 Significance and Innovation

The proposed research is particularly pertinent to Australia given the significant increase in the number of multi-storey buildings, especially residential structures, and it is obviously applicable more broadly.

In Australia, building construction accounts for 63% of the total construction industry, which is 5.4% of the total GDP (Australian Bureau of Statistics 2000).

Studies of recent building damage and failures have highlighted that damage or failure of non-structural components is by far the greatest contributor to financial losses and hence insurance claims (NAHB Research Centre 1994). While statistical cost data for non-structural damage is scarce, it is widely agreed and reported that the economic effects of all non-structural damage combined generally exceed those of structural damage in most earthquakes (Brunsdon and Clark 2001). In a survey of 355 high-rise buildings after the 1971 San Fernando earthquake, it was shown that in dollar value terms, 79% of the damage was non-structural (Arnold et al. 1987). The argument for explicit consideration of non-structural damage is even stronger in low to moderate seismicity regions, where the likelihood of significant structural damage is low and the response of buildings and their components are characterised by resonance and low damping.

It has also been found that serviceability failure of building components creates greatest havoc in terms of injuries and potential fatalities particularly in hospitals and other public buildings (Monto 1996). Consequently, the fundamental evaluation of the behaviour of the integrated non-structural components, and the skeleton of the building, is of significance to:

- Owners, operators and insurers of buildings. Valuable contents exacerbate the significance;
- Occupants of buildings and the general public; and
- Building product suppliers, manufacturers, designers and constructors.

In Australia, a great number of multi-storey buildings consist of either a reinforced concrete or steel skeletal structure with composite floor slabs and precast concrete or glazed façades with subsequent internal fit-out of light framed partitions. This project focused on this type of structure. Moreover, even though there are various types of non-structural components which work closely with the primary structure and are also of great importance to the structural performance, such as stairs, mechanical fits and services, etc., considering the diversity of the individual components in different buildings and the uncertainty existing in their construction techniques, façades and partition walls were chosen as the typical non-structural components analysed in this study.

The use by designers of finite element computer programs to analyse the skeletal structure of such buildings is now quite common. However, this design technique has not been extended to the complex issue of partial integration due to a lack of acceptable processes to address how a partial integration analysis may be achieved. This project used available sophisticated and enhanced finite element programs and computing capacity to modify the way in which traditional buildings are designed.

This application considered multi-storey buildings having more than six stories with façades and partitions. This form of structure was chosen as the main focus for the following reasons:

- The failure of façades and partitions may have drastic consequences with falling debris causing hazards both within and outside buildings particularly in areas with high density living conditions;
- Partitions often extend from the floor to the ceiling for fire rating and acoustic performance, hence making them vulnerable to differential movement between floors (inter-storey drifts);
- Façades and partitions may significantly alter the lateral behaviour of buildings as they substantially increase the stiffness and damping (Freeman 1977). Hence, taking both components into account may reduce the overall deflection, but increase the building natural frequency and base shear forces (Gad and Duffield 2000) and influence the distribution of damage;
- Façades can be up to 15% of the total building cost. They transfer significant wind pressure, and may experience large differential movement associated with inter-storey deformation and yet they rarely attract significant design and detailing attention.

It is envisaged that this new approach would provide practical integrated system design guidelines and limitations. More specifically, the research should lead to:

- Establishment of damage level for typical non-structural components and their systems for the type of multi-storey building discussed in this study;
- Development of rigorous analytical procedures for the integration of non-structural components in the design process;

- Improved understanding of actual building displacements. This is essential for the correct functioning of various electronic devices that require accurate spatial alignment: such technologies are increasing by being incorporated in modern high-rise buildings.

The proposed approach conforms to the current component design approach for ultimate conditions but clearly departs from this approach to account for serviceability conditions. It therefore concentrates on assessment of damage to non-structural components and contribution to the structure stiffness rather than the ultimate failure of the structure. Such changes may lead to potential cost savings. It is of direct application to structural engineers, practitioners, property owners, insurance and re-insurance companies and operators of important facilities such as hospitals and government control centre.

Current available design tools are primarily concerned with the structural skeleton. This study aims to provide a simple tool to assess potential contribution of non-structural components and damage to these components for new and existing buildings.

1.4 Research Methodology

To steadily and precisely approach this study, a comprehensive plan has been developed, involving various analytical methods. Figure 1-1 is a flow chart outlining the holistic approach to this study.

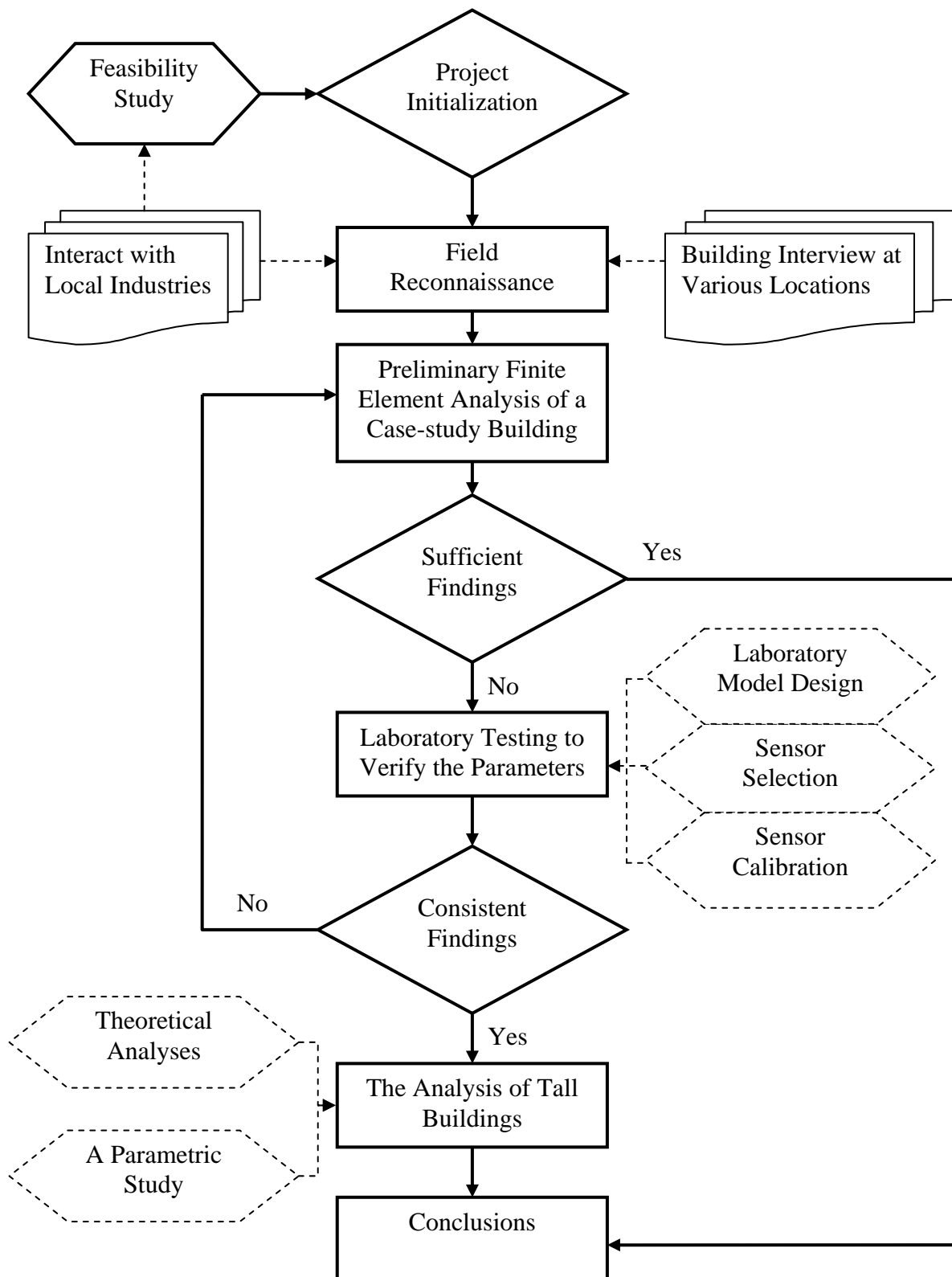


Figure 1-1 Flow chart of the holistic method of the study

In order to verify the feasibility of this study, broad discussions with local industries and investigations on different types of buildings have been conducted. It revealed that this project could be of significant importance to the Australian building and construction industry.

With the confirmation of the necessity, field reconnaissance was carried out. A case-study building provided by Bovis Lend Lease Pty. Ltd. was investigated and reviewed in detail. Finite element models were developed to incorporate the integrated structural analysis of the building. This identified the gaps between the building performance predicted by the structural analysis, integrating different non-structural components, and by the traditional analysis broadly adopted by the building industry.

The results obtained from the field investigation and the case-study building analyses were treated as preliminary findings with approximate accuracy.

Laboratory tests were designed to quantify the parameters identified in the field investigation. The testing results were used as input to the parametric study conducted using the finite element analyses.

Throughout the parametric study, evaluation of the contribution of different non-structural components to the overall building performance was established. Meanwhile, the damage level of individual components was identified.

1.5 Overview of the Thesis

This thesis contains 8 chapters. Chapter 1 is the introduction.

Chapter 2: Literature review. In this chapter, the development of construction materials and the lateral resisting systems of tall buildings are reviewed. The review of both relevant local and international standards is also included. To demonstrate the critical influencing factors of the lateral performance of tall buildings, the literature relating to different elements, both structural and non-structural elements, as well as connections were carefully examined. Further, the relevance of up-to-date finite element modelling and analysis methods was investigated. Based on the review, the gap between the current research status and the real performance of the multi-storey buildings was identified.

Chapter 3: Research Methodology. As explained in Section 1.3, a detailed plan was developed to conduct this research. The purpose of this study is to evaluate the effects of non-structural components on the overall structural performance. To achieve the aim, comprehensive analyses were conducted, a case-study building was investigated and the key non-structural components and sensitive parameters were identified. In order to

calibrate the parameters and validate the preliminary results drawn from the analysis of the case-study building, laboratory tests were carried out on a scaled model, the design of which was based on the scaling theory from Sabnis (Sabnis 1983). Based on the information collected through both the field reconnaissance and the laboratory tests, a parametric study was developed, to quantify the effects of different non-structural components on the overall structural performance as well as the damage level of individual components.

Chapter 4: Field Reconnaissance. Fifteen buildings in different cities in the Asia-Pacific region were investigated to obtain in-depth understanding of their behaviour. Based on the investigation of the different types of buildings, comparisons and some concluding remarks were drawn. Through the field reconnaissance, a thorough understanding of design features and performance of buildings in different regions was gained, identifying the gap between practice and design analysis.

Chapter 5: Preliminary Finite Element Analysis of a Case-study Building. A case-study building provided by Bovis Lend Lease Pty. Ltd. was investigated with focus on the interactions between non-structural components and the primary structure. Finite element models based on the field reconnaissance were developed and further analyses were carried out. Preliminary conclusions on the effect of different non-structural components on the performance of building system were drawn from the field reconnaissance and the corresponding finite element analyses, together with the identification of critical parameters.

Chapter 6: Laboratory Testing. The laboratory testing was to calibrate and validate the parameters identified in the previous chapter. A 1:100 scaled model of a typical 30-storey tall building was designed using the scaling theory from Sabnis (Sabnis 1983). In the test, various sensors were selected and calibrated to identify the most suitable device for the specific model developed in this study. A 4-stage test was carried out, to identify the influence of non-structural components on the structural performance. Detailed analysis and discussion of the test results are also provided.

Chapter 7: The Analysis of Tall Buildings. It includes two parts: the theoretical analyses and a parametric study. In this section, a detailed investigation of the influence of integrating non-structural components into the structural analysis of the primary structure was conducted. A parametric study of the influence of different non-structural components on the structural performance, based on the theoretical and finite element

analysis is discussed. A typical steel-frame building with concrete service cores was analysed. Different non-structural components such as façades, infill walls, etc. were included. From the analyses, the reliability of the finite element models was validated, and the contributions of non-structural components to both the serviceability and the strength of the structure were quantified. Simultaneously, the effects of different non-structural components were evaluated and hence the damage levels associated with them were identified.

Chapter 8: Conclusions.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

High-rise buildings are in high demand because of the world population boom and developments of technology during past decades. Various structural forms and construction materials were developed along with a diverse assemblage of structural and non-structural components. The real performance of the high-rise structures depends greatly on the integrated interaction of structural and non-structural components. This makes the behaviour of high-rise structures complicated.

In current design practice, the lateral load resisting system of a high-rise building is considered vital to the whole structure. It is commonly recognised by engineers that the load resisting system of a multi-storey building is a system including mainly structural components such as columns, beams, and shear walls. In fact though, non-structural components also contribute to the lateral performance of high-rise buildings.

Observable gaps exist between the real performance of buildings and the behaviour predicted by design theory. In practice, buildings perform as an integrated system of structural and non-structural components but the non-structural components are considered non-load bearing and are not included during the design process of the primary structure.

To obtain an understanding of tall building structural behaviour, a detailed review of the development of high-rise buildings, both from the structural form perspective and the material perspective are provided in this chapter. Further more, detailed discussion of individual elements, both structural and non-structural, is developed based on appropriate literature. A comprehensive understanding of tall building structures and the principles of their behaviour are established.

In terms of current design practice, a review of current standards and construction practice are included in this chapter, with the focus on various non-structural components. It again reveals that gaps exist between the design practice expectation and the actual building performance.

Difficulties always exist in measuring real performance of high-rise structures. Highly developed GPS measurement technology provides great opportunities for the structural measurement of high-rise buildings. Compared to traditional measurement devices such as accelerometers, the advantages offered by GPS measurement technology can be significant in the design process. However, because of testing constraints and laboratory conditions, traditional measurement systems are still preferable for structural models.

This chapter reviews the design of high-rise buildings and current design considerations. Aspects influencing lateral behaviour of high-rise buildings have been summarised together with the review of standards widely used in Australia and abroad. A brief discussion on structural measurement techniques is provided as preparation for the evaluation and selection of the measurement system for this study. Similarly, different analytical methods were investigated and structure modelling techniques were compared.

2.2 Design Overview of High-rise Buildings

2.2.1 Overview of Design Development of High-rise Buildings

A high-rise building is commonly defined as a multi-story building which is tall enough to be affected by lateral forces such as wind, earthquakes or blasts to the extent that these forces become critical design factors for the structural system (Stafford Smith and Coull 1991). With developing of economies and world population growth, more and more tall buildings for commercial and residential purpose are being built. From the first high-rise building in the middle of 19th century to the recently constructed world tallest buildings, for example Taipei 101 and Burj Dubai in Dubai, the design of high-rise buildings has experienced various stages of evolution to provide stronger, safer, more efficient, and more comfortable structures. In order to understand the development of high-rise buildings, as well as the current design practice for tall buildings, a detailed review of materials and structural forms of development for high-rise buildings is provided in this section. A review of current design procedures follows in the next section.

2.2.1.1 Development of Principle Construction Materials

Figure 2-1 explains the development of structural materials. As shown in the figure, timber and masonry were the two most commonly used construction materials in the

past. However, to build higher structures and to overcome the disadvantages of these two materials (such as lack of strength, fire problems and high self-weight) engineers have investigated various other materials and also improved their design methods and construction techniques. Eventually, steel became available for tall building construction. Its advantages included high strength, relatively low self-weight and high ductility. These properties ensured that issues relating to height and safety could be properly addressed in the relevant building codes. The 9-story Rand-McNally Building was the first all-steel framed tall building built in 1889. The Empire State Building used braced steel frame to achieve the height of 1250 ft and for many years it represented the crowning glory of the skyscraper construction. Also, steel was preferred for its constructability and reliability in terms of connections. Welding and bolting were both readily applicable methods for connecting steel members.

Initially, compared with steel, reinforced concrete was not as popular in high-rise construction. The process of its development was slow and the inherent advantages of the composite material were not fully appreciated (Stafford Smith and Coull 1991). Today, together with steel, reinforced concrete becomes one of the major construction materials of high rise buildings because of its high capacity both in compression (taking by the concrete) and in tension (taking by the reinforcing steel bars). Also a certain amount of ductility is provided by the reinforcing steel bars.

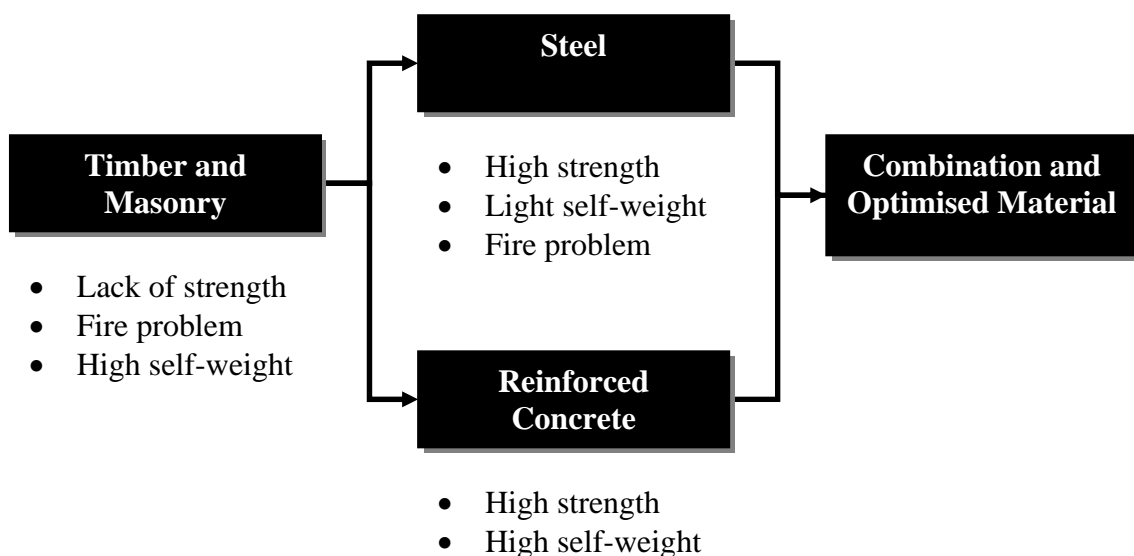


Figure 2-1 Development of construction materials

2.2.1.2 Development of Structural Forms and Lateral Load Resisting Systems

Advanced materials provide a chance of getting higher and safer structures, whilst improved structural forms provide aesthetic advantages. Table 2-1 summarises different forms of high-rise structures. It indicates that technically, the height of a building is based mainly on the structural form and the construction material. When a building is tall enough, movements induced by lateral loads such as wind and earthquakes become significant.

The design of high-rise structures has been developed according to requirements of the overall building strength and stability, as well as the needs of clients in different circumstances. Generally, from a design point of view, the concept of the structural system for a narrow tall building is considered as a beam cantilevering from the earth (Taranath 1998). Dictated by various needs of clients and construction technology, the structural form of tall buildings has developed from braced truss systems to the hybrid structures. To date, even though the criteria for the design of high-rise buildings vary across different design codes, the basic concept of the design of high-rise building is to provide the building with sufficient strength, stability, and serviceability. Using this basic concept, consideration is given to load resisting systems and stiffness and serviceability requirements such as drift limitations, human comfort criteria and fire protection, etc..

Table 2-1 Development of high-rise structures

Form	Lateral Force Resisting Members	Advantages	Preferred Material	Height tolerance
Braced-Frame Structure	Diagonal members girders, vertical truss, columns	<ul style="list-style-type: none"> • Highly efficient in resisting lateral loads • High stiffness for a minimum of additional material 	Steel	Very tall
Rigid-Frame Structure	Columns, girders, and connections	<ul style="list-style-type: none"> • Open rectangular arrangement allows freedom of planning and easy fitting of doors and windows 	Reinforced concrete/ Steel	Up to 25 stories
Infilled-Frame Structure	Infill	<ul style="list-style-type: none"> • Hard to predict the stiffness • Complex interactive behaviour 	Reinforced concrete/ Steel	Up to 30 stories
Flat-Plate and Flat-Slab Structure	Similar to rigid-frame structure, and flat-plate or flat-slab	<ul style="list-style-type: none"> • Simplest and most logical structure form 	Reinforced concrete	Up to 25 stories
Shear Wall Structure	Continuous vertical walls	<ul style="list-style-type: none"> • Very high in-plane stiffness and strength 	Reinforced concrete/ Masonry	Up to 35 stories
Wall-Frame Structure	Shear wall and rigid frame	<ul style="list-style-type: none"> • Stiffer and stronger 	Concrete	40 to 60 stories
Framed-Tube Structure	Tube formed by very stiff moment resisting frames	<ul style="list-style-type: none"> • High structural efficiency • Appropriate for use up to the greatest of heights 	Steel/ Reinforced concrete	40 to more than 100 stories
Outrigger-Braced Structure	Outrigger trusses, braced core, columns	<ul style="list-style-type: none"> • Efficient • Provides greater construction height • The ground floor can be entirely free of major vertical members 	Steel/ Reinforced concrete	40 to 70 stories
Suspended Structure	Cantilever trusses, braced core, hangers	<ul style="list-style-type: none"> • Some construction advantages • Similar to the suspended structure 	Steel/ Reinforced concrete	Not specified
Core Structure	Single core	<ul style="list-style-type: none"> • Inefficient in resisting lateral loading and supporting the floor loading 	Steel/ Reinforced concrete	Not specified
Space Structure	A three-dimensional triangulated frame	<ul style="list-style-type: none"> • High efficient • Relatively light weight • Potential to achieve the greatest height 	Steel/ Reinforced concrete	Not specified
Hybrid Structure	Combination of two or more above structure forms			

2.2.2 Overview of Current Design Philosophy of High-rise Buildings

To thoroughly understand building performance, it is essential that current design philosophy is fully understood. From World War II, the design philosophy and code formats shifted their emphasis from “the earlier working stress or ultimate strength deterministic bases to modern more generally accepted probability-based approaches”. This approach aims to ensure the structure and its constituent components are designed to resist the worst load case and deformations during construction and service, with proper safety and adequate durability during their lifetime (Stafford Smith and Coull 1991). There are two limit states: ultimate limit state and serviceability limit state. They govern the performance of the entire structure, and any part of it.

2.2.2.1 Ultimate Limit State and Serviceability Limit State

The ultimate limit state concerns loads causing structural failure, including instability (Stafford Smith and Coull 1991). Design of tall buildings in resisting the ultimate limit state loads means designing the load resisting system of the building to resist both vertical and lateral loads which could induce catastrophic effects on the building.

The serviceability limit state is more related to structural vibrations, horizontal and vertical deflections, structural cracking and the compatibility of these issues with the secondary elements supported by the structure, such as partitions, cladding, finishes, etc. (Blake 1989). Serviceability design of a building is based on normal wind and moderate earthquake conditions. Until recently blast loading was normally not considered. If unsuccessful in fulfilling either of these two limit states, the structure or the components are considered to have “failed”.

2.2.2.2 Currently Accepted Design Philosophy for High-rise Buildings

It is widely accepted that the primary structure is the load-bearing system composed of columns and cross members, only capable of transferring vertical loads, while lateral loads would be transferred via beams and panels (Eisele and Kloft 2003). Moreover, in the current design practice, the building is always designed with its primary structure as the load bearing system whilst the non-structural components are not considered in the structural design.

The review of previous research indicates that, it is commonly agreed that the building is an integrated system including inter-connected structural and non-structural

components. Thus, it can be concluded, that the general load bearing and distribution system of a high-rise structure should be as shown in Figure 2-2.

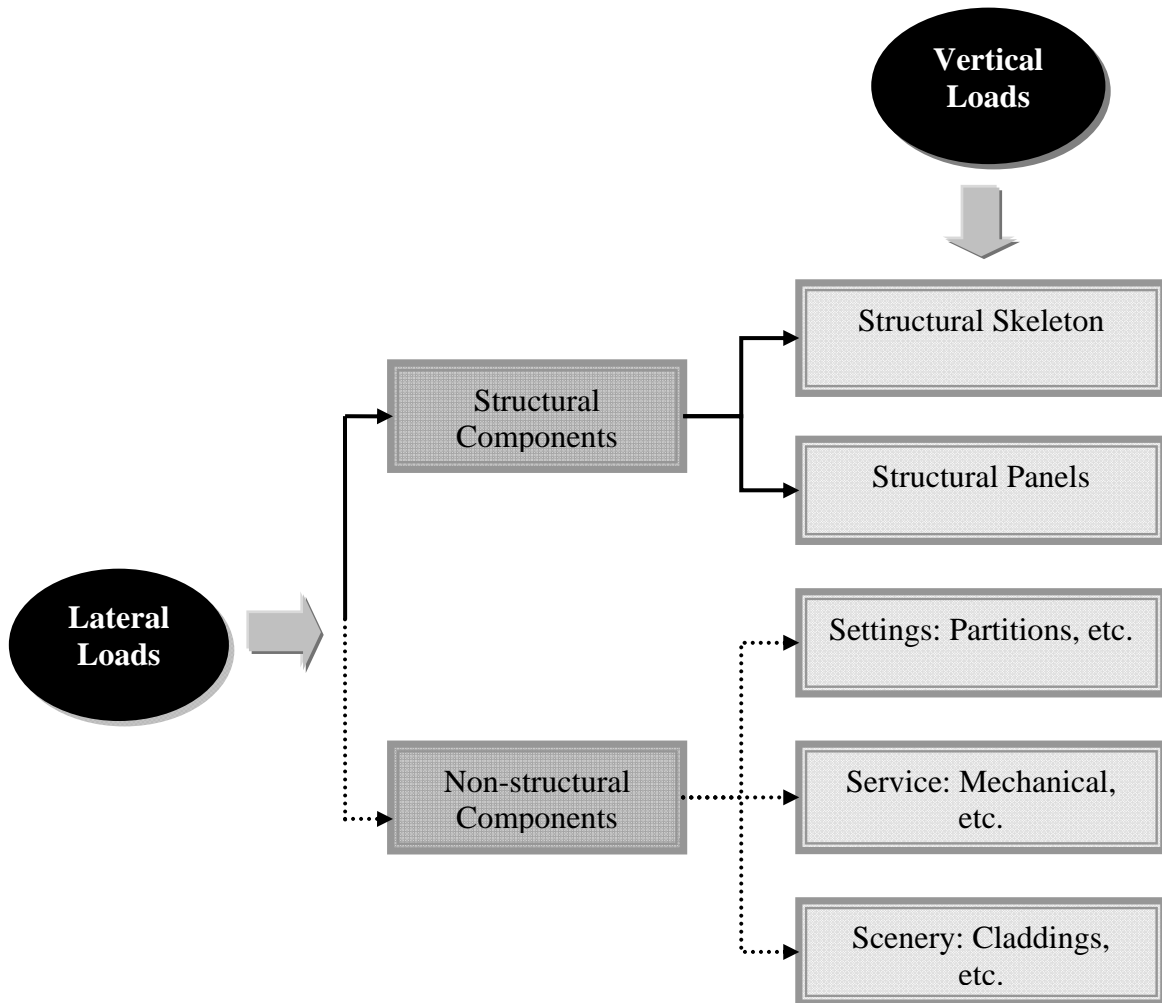


Figure 2-2 Load distribution chart of high-rise buildings

Most modern design standards and building codes are based on the dual design philosophy of the two limit states. Design of the primary structure under the two limit states has been given the first priority in the design of buildings. A detailed review of current design standards is provided in Section 2.4.

In terms of high-rise buildings, since they are always treated as vertical cantilever structures in the structural analysis (Stafford Smith and Coull 1991), it is easy to understand that the higher the building, the more important the lateral behaviour. Thus, to understand the performance of high-rise buildings, the lateral resisting system of tall buildings becomes a key factor that needs to be investigated and understood.

2.3 Further Investigation of Lateral Performance Influencing Aspects

As identified in previous sections, increase of the height of a building means that lateral stability becomes an important factor to be considered in design process. Based on the above dual design philosophy, research has been conducted focusing on the design of lateral resisting systems for tall buildings. As shown in Table 2-1, lateral load resisting systems for high-rise buildings may be described in various forms associated with different structural configurations.

Figure 2-3 summarises different lateral resisting systems currently in use. In summary, rigid framed structures, frames with shear walls, frames with service cores, frames with bracing systems, and tube in tube structures, etc. are now widely used as typical primary structural systems in the design of high-rise buildings (Stafford Smith and Coull 1991; Beedle and Rice 1996; Choia et al. 2004; Strelitz 2005).

The components in different primary structural systems for high-rise buildings (beams, columns, shear walls and cores) are the main elements which are treated as structural components in the building system and are designed according to design standards (e.g. AS/NZS 1170:2002).

However, Gad (Gad et al. 1999) pointed out that the lateral behaviour of framed residential low-rise structures might well be influenced by roof diaphragm behaviour, wall panel behaviour, and veneer frame interaction (Figure 2-4). Considering different components in high-rise structures, similar to the above theory, an extrapolation can be made. That is, the lateral loads on a tall building are resisted by a lateral resisting system composed of the primary structural system, i.e. the frame plus shear wall panels, the roof system, etc.. The “non-structural components” may include infill walls and façades, etc.. The interaction of structural and non-structural components is shown in Figure 2-5.

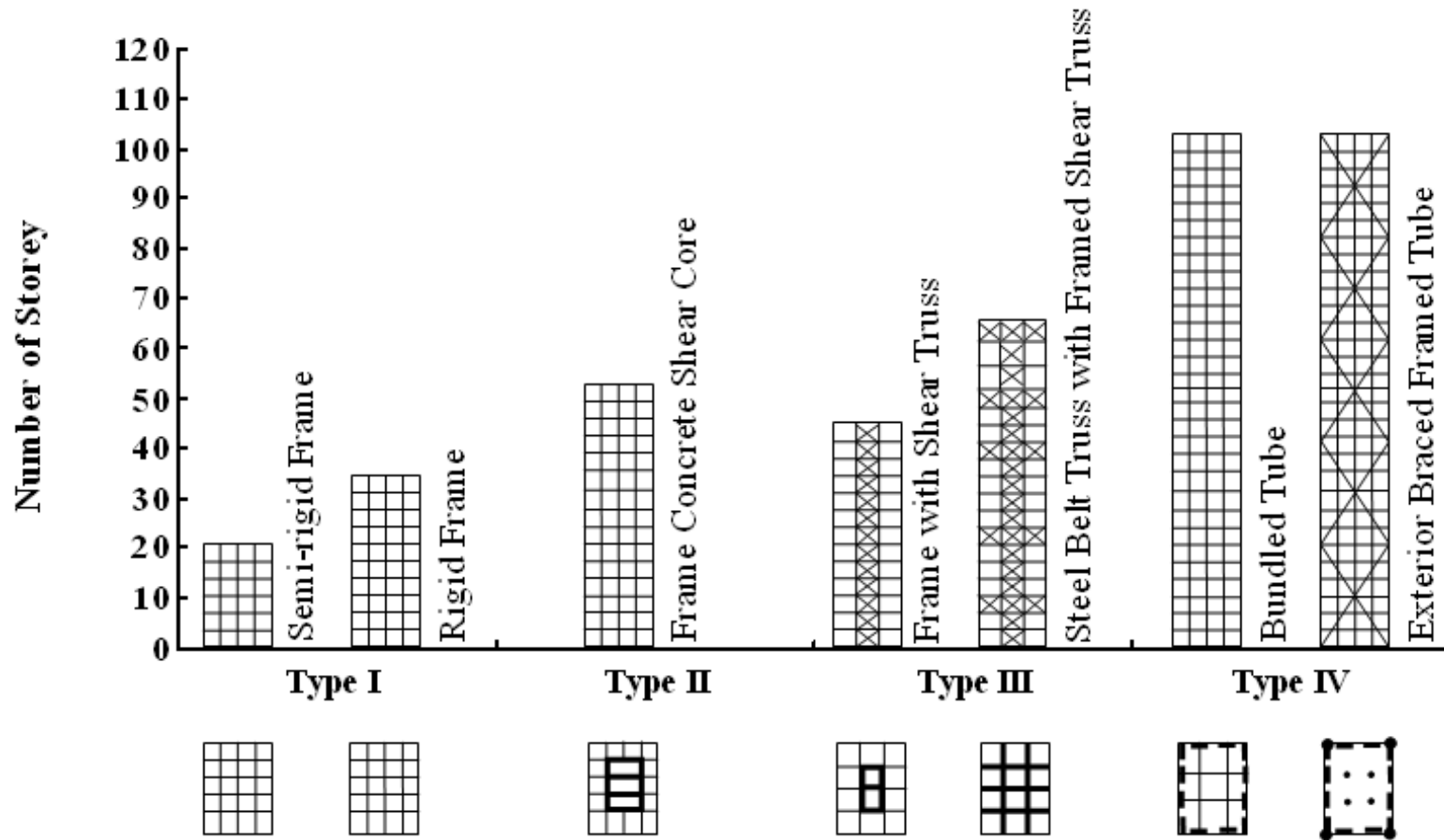


Figure 2-3 Development of structure expression

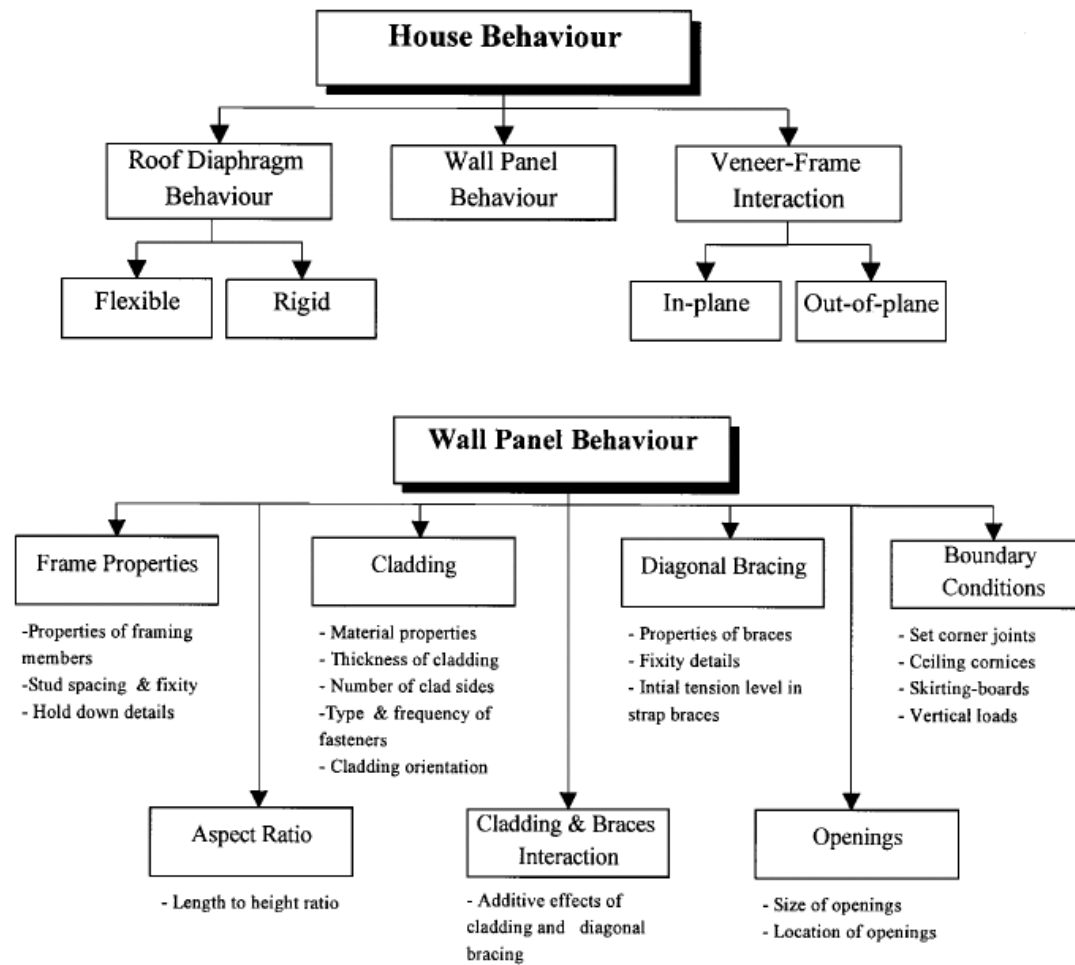


Figure 2-4 Behaviour of a typical one-storey house (Gad 1997)

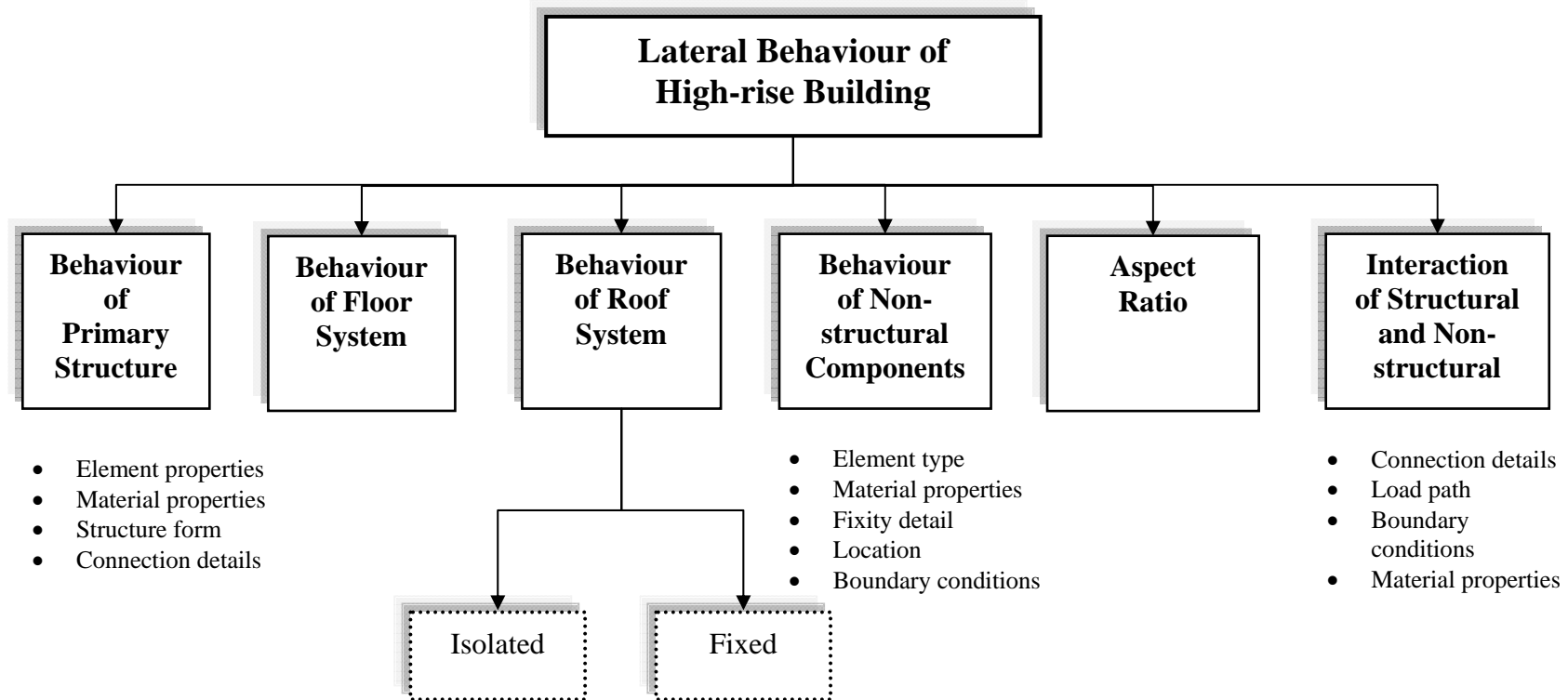


Figure 2-5 Influential aspects of lateral behaviour of high-rise buildings

2.3.1 Primary Structural System

Various high-rise building designs and concepts have been discussed in Section 2.2.1. In tandem with the development of these structures, different types of primary structural systems have also been developed (Figure 2-4). Generally, the main components of the primary structural system are, as listed in the previous section, the structural frame composed of columns and beams, the structural wall system, the floor system and the roof system. Detailed discussion concerning these components is provided in the following sections.

2.3.1.1 Frame

Steel and concrete can be both adopted as the main material for the structural frame of a high-rise building. Generally, a high-rise building having a rigid frame as its lateral load resisting system is referred to as a framed structure. Columns and beams are the main components of a structural frame and are rigidly connected by reinforcement steel bars (concrete structure), bolts (steel structure) and welding (steel structure).

Deierlein (1997) summarised the behaviour of steel-framed structures. A detailed description used for understanding the steel material used for structural frames was provided. For reinforced concrete (RC) frame, Kirke et al. (2004) estimated the failure probabilities of RC frame structures by analysing earthquake damage to buildings in Singapore and modelling typical low, mid and high-rise structures respectively. The study showed that, the inter-storey drift of low, mid, and high-rise models was 0.1%, 0.92%, and 1.3%. This means that in terms of hazard such as the far-field earthquake in Sumatra and in the offshore trench, with the increase of the building height, the lateral performance of structures (represented here by inter-storey drift) under seismic load becomes increasingly critical.

Further, in terms of efficiency of the frame as a lateral resisting system for tall buildings, the internal frame is relatively inefficient and flexible and is always restricted by the column to column space and limited depth of the spandrels. However, the external frame can be designed to have closely spaced columns connected by deep spandrels and the entire perimeter of the building could be efficiently developed as a “boxed frame” or “framed tube” to resist lateral loads (Blake 1989).

Braced frames and frames with outriggers are other efficient systems for high-rise framing. When single or double-diagonal braces or K-bracing are used in a beam and column framework or internally around a service core or in an external wall, the major vertical loads are transferred to the corner columns and the overall stiffness of the structure is increased. Cohen (1986) pointed out that the staggered-steel-truss system was efficient and it was typically used in 15- to 20-storey buildings. The main structural benefit and resultant efficiency were that the system's resistance to lateral loads acts in parallel with the trusses. Moreover, according to Sabelli et al. (2001), individual braces often possess only limited ductile capacity under cyclic loading. Design simplifications and practical considerations often result in the brace selected for some stories being far stronger than required and a buckling-restrained bracing system which can efficiently overcome many potential problems of special concentric braced frames has been introduced. Buckling-restrained braces exhibit the same load-deformation behaviour in both compression and tension and have a higher energy absorption capacity which is easily adjustable for both stiffness and strength (Xie 2005). It was observed that non-buckling bracing is effective in dissipating energy and controlling inter-storey drift (Ravi Kumar et al. 2006).

In summary, various studies that have been carried out investigating and analysing the behaviour of structural frames in high-rise buildings are mature and well understood. It has been demonstrated that even though rigid frame systems are efficient in resisting lateral and vertical loads when being used in the multi-storey buildings, with an increase of the height to base width ratio, an increasing number of bracing systems have been introduced in the design of framed structures. They have demonstrated their efficiency in improving the overall building performance, especially under ultimate loads.

2.3.1.2 Structural Wall Systems

“Shear walls may be internal or external or may surround internal service areas to form cores. Their location and dimensioning are major design elements since they seriously impinge on internal planning and may affect external appearance.” (Blake 1989)

If shear walls should be used alone, three general requirements need to be fulfilled (Blake 1989):

- (1) *At least three shear walls must be provided of which at least two must be parallel and widely spaced, to provide torsional resistance, with the third at right angles;*
- (2) *The centroid of the shear walls should be close to the centre of gravity of the loading;*
- (3) *Walls likely to need very large openings should be avoided if alternatives are available, since their stiffness and, hence, load-resisting contributions will be diminished substantially.*

Many high-rise structures were designed as structural frames with shear walls that can effectively resist horizontal forces. In the Asian region, most recently constructed high-rise apartment buildings have adopted a box structural system that consists of reinforced concrete walls and slabs (Kim et al. 2005).

In some office buildings, where open space is usually an important requirement, and, because of flexibility demands in its special design, internal bracing walls should be avoided where possible. Generally, external bracing walls are used, in conjunction with internal cores (Blake 1989).

Ekwueme et al. (1997) analysed the efficiency of flanged walls (L-, T- or C-shaped walls) in resisting lateral loads in the tall building design. It was noted that flanged walls typically have different strength, stiffness and ductility capacities in the two orthogonal directions when loads are parallel to the web. They are critical in resisting various lateral loads applied to tall buildings. Analyses showed that buildings with flanged walls incur less structural damage from moderate earthquakes than buildings without flanged walls. However, significant structural damage in buildings with flanged walls would occur in a major earthquake because of the large ductility demands. Non-structural damage in flanged-wall buildings was about 20%-40% greater than that in building without flanged walls.

Dedolph Jr. et al. (1997), Griffith and Alaia (1997), Liew et al. (2002), Doherty et al. (2002), and Ajrab et al. (2004), considered the analysis and optimisation of various wall systems to resist lateral loads. The capability of wall systems in high-rise buildings to resist lateral loads has been improved substantially. The identification of the importance of wall systems in resisting lateral loads in tall buildings has illustrated that problems relating to racking capability, inter-storey drift, and out-of-plane bending could be successfully reduced or even overcome by using such wall systems.

Hoenderkamp (2004) identified that for outrigger truss-braced high-rise shear wall structures, six parameters, bending stiffness of the shear walls, bending and racking shear stiffness of the outriggers, overall bending stiffness contributions from the exterior columns, and rotational stiffness for shear walls and the column foundations should be considered in the preliminary analysis to decide the optimum location of the outriggers. Horizontal deformation and internal forces in the structure could be significantly reduced by increasing the racking shear stiffness of the outriggers and the stiffness of the foundations of the exterior columns.

Some computational methods have been developed in order to model and calculate the contribution of wall systems to the overall performance of high-rise buildings. Li (2001), Lu and Chen (2005), and Kim et al. (2003; 2005) introduced flexural-shear plate models, non-linear macro-models and super element models respectively. These models not only efficiently identified the accurate solutions for non-linear and buckling analyses of shear-walls, but also contributed to great savings in computational time.

In summary, if being used together with frame or core systems, shear walls are an efficient lateral resisting component for high-rise buildings. However, working alone as the lateral load resisting system for tall buildings, shear walls may greatly constrain the height of the building because of their own stiffness and strength limitations. Research has indicated that, shear walls with openings, especially large openings, should be used with great caution due to the reduced strength and stiffness caused by the openings.

2.3.1.3 Floor Slabs

The floor slab is also an important structural component of a building. It not only connects the main structural components to form a box system or rigid diaphragm, but also provides flexural stiffness to the overall structure. However, the flexural stiffness of slabs is generally ignored in the analysis of high-rise building structures. Lee et al. (2002) pointed out that it might be reasonable in the analysis of framed structures, but in box system structures, the floor slabs may have a significant influence on the lateral response of the structure. A finite element model comprising super elements, frictional beams and floor slabs were validated by Lee et al. (2002) as reliable and capable of predicting the real performance of buildings after comparison with relevant test data. Also, Horr et al. (2003) introduced a new individual grid-stiffened flooring system (IGSF) together with a multi-layered frame system to resist the initial impact forces and progressive collapse phenomena. The results from these analyses illustrated that the

flexural stiffness of floor slabs made a great contribution to resisting lateral loads in high-rise buildings and should not be ignored in design process.

Based on the above discussion, when conducting the analysis of the lateral performance of a tall building structure, floor slabs should be integrated and considered as an important component in the lateral resisting system.

2.3.1.4 Roof System

The situation is similar but not exactly the same for roof systems. The roof system of a structure also has great influence on the overall performance of the structure in that it can increase the stiffness of the building if it is passively connected to the main structure.

According to Villaverde (2000), if the roof system is isolated from the high rise structure, lower rotational ductility demands in beams and columns, as well as lower inter-storey drift ratios would be achieved. Thus, the response of buildings to earthquake would be reduced.

Tamura et al. (1996), He and Song (1997), Ahmad and Kumar (2001), Letchford et al. (2002), and Fu et al. (2006), analysed the different behaviour of roof systems. The dynamic characteristics and interference effects under wind loads on roof systems were identified. These observations helped designers take the influence of roof behaviour into consideration when analysing the lateral performance of buildings. The prediction of building performance under lateral loads, especially wind loads was hence more accurate and realistic.

In this section, individual components, such as frames, walls, floor slabs and roof systems, which may influence the lateral performance of tall buildings, are discussed. Based on the review, it is clear that these elements are of great importance to the lateral behaviour of high-rise buildings. However, to achieve the maximum efficiency, detailed analysis and optimisation processes need to be conducted. These analyses are to a great extent on a case-to-case basis.

Moreover, when coming to a systematic review of building performance, another group of components, categorised as “non-structural components”, has attracted a significant amount of attention from researchers worldwide. Discussion on this type of component is provided in the next section.

2.3.2 Non-structural Components

2.3.2.1 Definition and Classification of Non-structural Components

Non-structural components are those attached to or housed in a building or building system, but they are not considered part of the main load resisting structural system of the building (Massey and Megget 1992; Mondal and Jain 2005).

There are three types of non-structural components,

- *Architectural components, such as parapets, veneer, cladding systems, suspended ceiling, penthouses, etc.;*
- *Mechanical components, such as boilers, storage tanks, piping systems, fire protection systems; and*
- *Electrical components, such as computers, data acquisition systems, electric motors, light fixtures, etc..*

Depending on different performance, non-structural components can be classified as deformation or acceleration sensitive. If the behaviour of a non-structural component is controlled by the deformation of supporting structure, measured by especially inter-storey drift, this non-structural component is deformation sensitive. Satisfactory performance of deformation sensitive components can be achieved in two ways: by limiting the inter-storey drift of the supporting structure, or by designing the component or system to accommodate the expected lateral displacement without damage. When the non-structural component is not vulnerable to damage from inter-storey displacements, it is acceleration sensitive. This type of component should be anchored or braced to the structure to prevent movement under the design loading. Many components are both deformation and acceleration sensitive while the primary mode of behaviour could generally be identified as shown in Table 2-2 (Naeim 2001).

Table 2-2 Classification of non-structural components

Component	Sensitivity		Component	Sensitivity	
	Acc.	Def.		Acc.	Def.
A. Architectural			B. Mechanical Equipment		
Exterior Skin			Mechanical Equipment		
Adhered Veneer	S	P	Boilers and Furnaces	P	
Anchored Veneer	S	P	General Mfg. And Process Machinery	P	
1 Glass Blocks	S	P	1 HVAC Equipment, Vibration Isolated	P	
Prefabricated Panels	S	P	HVAC Equipment, Non-vibration Isolated	P	
Glazing Systems	S	P	HVAC Equipment, Mounted	P	
Partitions			In-line with Ductwork		
2 Heavy	S	P	Storage Vessels and Water Heaters		
Light	S	P			
Interior Veneers			2 Structural Supported Vessels (Category 1)	P	
Stone, Including			Flat Bottom Vessels (Category 2)	P	
3 Marble	S	P	3 Pressure Piping	P	S
Ceramic Tile	S	P	4 Fire Suppression Piping	P	S
Ceilings			5 Fluid Piping, not Fire Suppression		
a. Directly Applied to Structure	P		Hazardous Materials	P	S
b. Dropped, Furred,	P		Non-hazardous Materials	P	S
4 Gypsum Board			6 Ductwork	P	S
c. Suspended Lath and Plaster	S	P			
d. Suspended Integrated Ceiling	S	P	Acc. = Acceleration Sensitive		
5 Parapets and Appendages	P		P = Primary Response		
6 Canopies and Marquees	P		Def. = Deformation Sensitive		
7 Chimneys and Stacks	P		S = Secondary Response		
8 Stairs	P	S			

2.3.2.2 Damage and Loss Caused by Failure of Non-structure Components

As discussed in the previous section, non-structural components are either deformation or acceleration sensitive, or both. Structural movements can lead directly to the damage of different non-structural components, especially some architectural non-structural components. Moreover, according to recent research (Naeim 2001), non-structural components take up a high percentage of the total capital investments of majority of buildings. Failure of these components can disrupt the function of a building as surely as structural damage, and can pose a significant safety risk to building occupants as well. Death, injuries and panic caused by the failure of non-structure components of buildings

are experienced by the public when disasters such as earthquakes (not even severe ones), hurricanes and blasts occur.

The results from research by Onur et al. (2004) have shown that after a number of recent earthquakes, damage to non-structural components constituted the largest portion of the monetary losses. In modern high-rise buildings, the cost of non-structural components represents over 50% of the overall construction costs (Hira 2002). Therefore, whilst a building may withstand an earthquake with little damage to its primary load resisting structure, the cost of repair of the secondary elements damaged internally could be a large percentage of the building original construction costs.

In the analysis of the 1989 Newcastle Earthquake (Melchers 1990), the following conclusions were drawn.

“In general, damage to modern buildings in Newcastle was confined to non-structural aspects and is broadly in accordance with structural engineering expectations, given that the buildings were not specifically designed to be earthquake resistant;

A large part of the damage observed in Newcastle is related to masonry construction. Much of this is non-structural. It is considered that the damage caused to modern construction would have been less had some low level structural engineering design requirement existed for lateral loading on masonry and other forms of non-ductile construction.”

2.3.2.3 Current Observations

With the development of lateral load resisting systems for high-rise buildings, more and more researchers (Melchers 1990; Arnold 1991; Hall 1995; Naeim 1999; Hoenderkamp and Snijder 2003; Su et al. 2005) noted the structural role of “non-structural” components, such as façades, staggered truss systems, partition walls, in resisting the lateral loads. It has been recognised that the overall performance of a building will be significantly influenced by the interaction between the structural skeleton and the “non-structural” components.

The actual performance of a building differs significantly from that of idealised structural models (Naeim 1999; Sugiyama 2000). Naeim (1999) identified that the seismic force demands experienced by several buildings, both at the roof and the base areas, exceeded the design force levels recommended by various codes and guidelines.

This indicates that, by including non-structural components, the real performance and load resisting capacity of the overall building system are shifted from what was designed, and should be thoroughly re-analysed to overcome the existing gaps between the theoretical analysis and the current practice.

Hoenderkamp and Snijder (2003) pointed out that by locating façade riggers at the end braced-frames in a high-rise structure, all columns in the two end-façade system would participate in resisting lateral loads. Gad et al. (1999a; 1999b) identified that plasterboard, combined with other non-structure elements, such as ceiling cornices, could resist 60% to 70% of the lateral load subjected by a low-rise building while the strap brace can only resist 30%~40%. Moreover, by adding only two extra studs at either side of the plasterboard cladding, the lateral capacity increased by almost 50% (Liew et al. 2002).

Gad et al. (1998; 1999a; 1999b) had clearly shown that non-structural components in low-rise buildings could increase the structural lateral stiffness and strength by more than 100%. Accordingly, Su et al. (2005) found that the contribution of non-structural elements to the overall stiffness of tall buildings in their case studies could reach as much as 87%. This helps account for the difference between the theoretical evaluation and the real performance of tall buildings. It is common that after a medium earthquake, the structural skeleton may still stand, while some of the non-structural components have already been seriously damaged.

Laboratory tests were conducted for low-rise structures by Gad et al. (1999a; 1999b). Computer-aided analyses and design optimisation methods such as finite element modelling, etc. were more widely adopted for high-rise structures because of their reliability and flexibility. Finite element analyses provide a cost-effective design method and better indication of structural performance than could be otherwise achieved (Grierson and Khajehpour 2002; Kicinger et al. 2005; Sarma and Adeli 2005; Wu et al. 2006). However, in reviewing previous research, it is noted that although thorough consideration of the detail of the structural skeleton, and even some of non-structural elements have been included, a full understanding of the influence of non-structural components on structural performance and the behaviour of the integrated building system as a whole were not sufficiently understood to predict the actual performance of real structures under lateral loads.

In conclusion, an increasing number of studies show that non-structural components are playing an important role in the overall building performance. The stiffness of a building and hence the lateral deflection and natural frequency will be greatly influenced by various non-structural components.

2.3.3 Interaction of Structural and Non-structural Elements

The design process for a typical 30 to 50 story building involves designing a skeleton to resist the ultimate limit state loads and the serviceability limit state loads, including allowances for extreme wind and earthquakes. Structural engineers design high-rise buildings by taking little account of (notionally) non-structural components such as partitions, façades, doors, windows, ceilings and mechanical services in the design process. However, buildings are widely recognised as a complex assemblage of both structural skeleton and non-structural components (Su et al. 2005). Non-structural components are considered by designers as infill or providing internal services based on the assumption that they are isolated from the skeleton.

The actual behaviour of a high-rise building is very complicated because of the conflicting requirements of diverse (structural and non-structural) building systems (Sev 2001). Thus, the traditional design approach, which only considers the structural skeleton in the analysis of a high-rise building, is not accurate enough to predict the actual performance of the integrated building system.

In almost all high-rise buildings, the so-called “non-structural components” are involved in the building performance and provide lateral resistance. Moreover, interactions between non-structural and structural elements significantly influence the overall performance of a high rise structure. Three dimensional analysis methods can help better understand the behaviour of high-rise buildings compared with traditional two-dimensional analyse. In various case studies (Sev 2001), several well-known high rise buildings from all over the world were used to demonstrate the benefits of, not only the integration between structural and architectural design, but also the integration of structural and non-structural components during the design process. These cases illustrated the importance of the role played by non-structural components in the overall performance of high-rise structures.

It is noteworthy that, during the development of design concepts, more and more designers have noted that the interaction between structural and non-structural components might have significant influence on structural performance.

To get a better understanding of the interaction between structural and non-structural components, it is critical that the behaviour of different types of connections in a building are analysed.

Connections in a high-rise building can be broadly classified into three types:

- Connections between structural components;
- Connections between structural and non-structural components;
- Connections of non-structural components

2.3.3.1 Connections between Structural Components

In a framed structure, the connections between beams and columns are always required to be rigid. However, it has been noted that actual connections between structural components, such as beam-to-column connections, are not perfectly rigid, and the properties of these semi-rigid connections have been investigated for several decades (Ahmed and Nethercot 1998; Calado and Lamas 1998; Gizejowski et al. 1998; Rodrigues et al. 1998; Schneider and Alostaz 1998; Shanmugam et al. 1998; Dissanayake et al. 1999; Shakourzadeh et al. 1999; Liew et al. 2000; Nethercot 2000; Kemp and Nethercot 2001; Neves et al. 2001; Olsen 2001; Simoes da Silva and Girao Coelho 2001; de Lima et al. 2002; Masarira 2002; Turvey and Brooks 2002; Chen et al. 2004; Langdon and Schleyer 2004; Lim and Nethercot 2004; Vigh and Dunai 2004; Cabrero and Bayo 2005; Cheng and Chen 2005; Raftoyiannis 2005; Yu et al. 2005; Bayo et al. 2006; Casafont et al. 2006; Fu and Lam 2006; Porcaro et al. 2006; Wald et al. 2006; Wan-Shin and Hyun-Do 2006; Zaharia and Dubina 2006; Chen and Du 2007; Kabche et al. 2007). Based on differences of construction materials, connections between structural components can be further categorised into bolted connection, welded connection, and reinforced concrete connection. Bolted and welded connections are normally used in steel structures and composite structures. Most of studies about connections were conducted within this area. Identifying the behaviour and properties of beam-to-column connections was one of the most popular objectives of these researchers.

To check the effect of joints on the stability of steel structures, Masarira (2002) investigated eight types of connections by both numerical analysis and finite element modelling. It was pointed out that inaccuracies in the assessment of the effect of joints on the stability of structure frames occur in most standards. Ignoring those effects could be uneconomical and gaps exist between the current practice (standards) and the real behaviour of building connections. Improved methods were also presented to recognise these gaps (Simoes da Silva and Girao Coelho 2001; Chen et al. 2004; Lim and Nethercot 2004; Yu et al. 2005; Bayo et al. 2006). Simoes da Silva and Girao Coelho (2001) and Bayo et al. (2006) developed simplified models based on a conventional analytical spring model for semi-rigid connections. This diminished the limitation imposed by the β factor (safety index) proposed in Eurocode 3 and the real sized model itself. Bolted moment connections and connections reinforced with lengthened flange ribs (shown in Figure 2-6) were suggested by Yu et al. (2005), Lim and Nethercot (2004) and Chen et al. (2004) to be efficient and practical in improving structural performance.

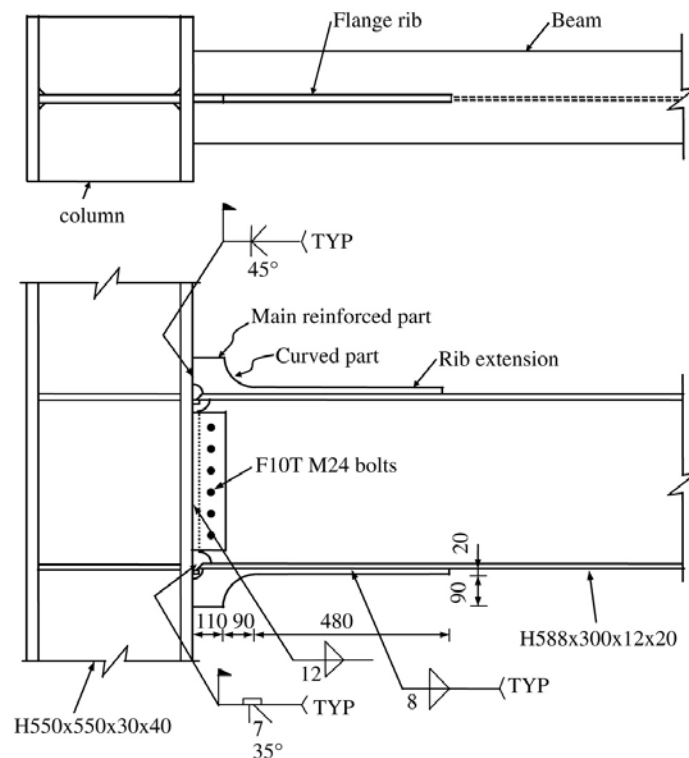


Figure 2-6 Typical rib-reinforced steel moment connection (Chen et al. 2004)

Some detailed methods for analysing and predicting the properties of connections were also identified by Dissanayake et al. (1999), Liew et al. (2000), Turvey and Brooks (2002), Kudzys (2006), Porcaro et al. (2006) and Zaharia and Dubina (2006). These researchers came out with findings concerning specific connection properties. When

subjected to bending moments, different failure mechanisms of bolted connections between steel I beams were detected. These included web crushing, bolt failure and uni-axial bending failure (Olsen 2001). According to Olsen, an increase of the endplate thickness with the same bolts increases the moment bearing capacity of the connection. Zaharia and Dubina (2006) analysed the stiffness of joints in bolt connected cold-formed steel trusses using a series of experiments. The results emphasized that the joint deformability was mainly due to the bearing load carried by the bolts. Further, there was only 2% difference in the ultimate load while there was a 37% difference in the corresponding displacements when the analysis considered both axial and rotational stiffness. Full-scale tests of steel-concrete composite connections have been conducted by Liew et al. (2000). These tests identified that composite connection properties had a close relationship with the reinforcement ratio, the steel element stiffening, and the concrete encasement. Properties of connections with fillet welds and self-piecing riveted connections were also discussed by Kudzys (2006) and Porcaro et al. (2006).

In summary, the connections between structural components have been well investigated. Improvements in the performance of structural connections have been dramatic.

2.3.3.2 Connections between Structural and Non-structural Components

As part of a building, non-structural components such as façades, infill walls, windows and doors, etc. play important roles in aesthetic, environmental and energy control aspects. Generally, they all have direct or indirect interactions with the primary structure through diverse connections such as bolting or welded connections. However, not so much research has been conducted to analyse the properties of these types of connections.

2.3.3.3 Requirements from Standards

According to the Australian Standard for Steel Structures (AS 4100), the following requirements of design of connections are specified.

When members subject to axial tension (Section 7):

- “When a connection is made by bolting or welding to all elements of the member cross-section, the member may be assumed to have a uniform stress distribution across the cross-section.” (Clause 7.3.1);

- “When the ends of members are connected such that not all elements of the member cross-section attached to the support, then additional stresses resulting from shear lag or eccentricity are induced and should be accounted for in the design.” (Clause 7.3.2);
- The design requirements of members with pin connections are “intended to prevent tearing-through at the end of the eye-bar and dishing of the plate around the pin.” (Clause 7.5). These provisions are summarised in Figure 2-7.

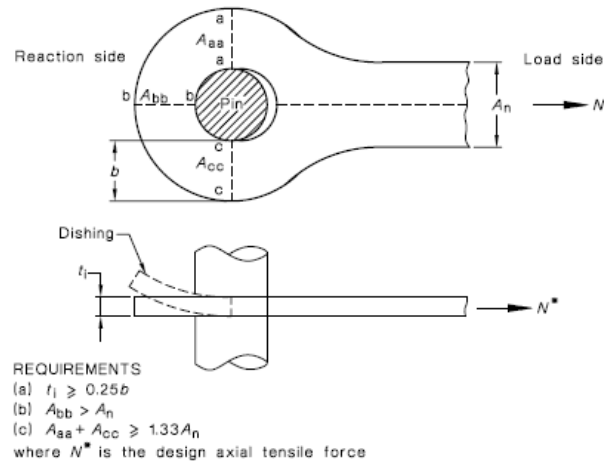


Figure 2-7 Design requirements of members with pin connections

Three forms of construction: rigid, semi-rigid, and simple (Clause 4.2).

- “It is important to note that practical connections are neither fully rigid nor fully flexible...semi-rigid connection design demands a knowledge of the true moment-rotation behaviour of the connection to enable a frame analysis to be carried out, and to allow the design of the connection itself;
- Practical simple connections will transit some bending moment to the supporting members...Loss of rigidity in a rigid connection will cause a redistribution of bending moments in a frame;
- The rotational behaviour of practical simple connections is most commonly provided for by allowing one or more elements in the connection to deform appreciably...”

Design of bolts, pin connections and welds (Section 9)

Details are listed in Appendix II.

2.4 Review of Standards and Codes

2.4.1 Introduction

Standards are living documents which reflect progress in science, technology and systems (AS/NZS 1170). To better understand the current situation of high-rise building design development, following standards were reviewed:

- AS/NZS 1170 Series
- AS/NZS 3600 Series
- FEMA Series
- EuroCode 8

The main focus of the review is the serviceability design (design requirements of non-structural components), and the interaction and integration identification requirements for structural and non-structural components.

2.4.2 AS/NZS 1170 Series (1993; 2002; 2007)

AS/NZS 1170 series is the standard for structural design actions. In this series, clear definitions of two limit states are given. Guidelines of both the ultimate and serviceability limit states have been discussed in detail. When introducing the load paths in AS/NZS 1170.0:2002, Section 6 (2002), declares that,

“The design of the structure shall provide load paths to the foundations for forces generated by all types of actions from all parts of the structure, including structural and non-structural components.”

It gives clear expression of the requirement to consider the load resisting and transferring capabilities of non-structural components.

In this section, whilst identifying the requirements for minimum lateral resistance of connections and ties, the standard requires *“all parts of the structure shall be interconnected”*. This again, emphasises the necessity for the integration of all the structural and non-structural components.

For the serviceability limit state, the standard suggests that it is generally best to deal with deflection design when the individual load is applied. A table of suggested serviceability criteria is given in its Appendix C, based on the annual probability of exceedance of 1/25 (beyond which serviceability issues have been observed). In that table, detailed deflection limits of single components are listed, including some of the non-structural components. It is noted that in terms of the non-structural components such as claddings, windows, façades, glazing system, the main actions that need to be considered are service level wind loading and earthquake loading. Regarding the deflection limits, detailed requirements on the non-structural components are listed in Table 2-3. This table is only a guideline and is imprecise, further investigation and verification should be conducted (AS/NZS 1170.0:2002).

Table 2-3 Serviceability limit state criteria for individual non-structural components from AS/NZS 1170:2002 (2002)

Element	Phenomenon	Serviceability Parameter	Applied Action	Element Response
Brittle cladding	Cracking	Mid-height deflection	W_s	Height/500
Masonry walls (in plane)	Noticeable cracking	Deflection at the top	W_s or E_s	Height/600
Plaster/gypsum walls (in plane)	Lining damage	Mid-height deflection	W_s	Height/300
Glazing system	Bowing	Mid-span deflection	W_s	Span/400
Windows, façade, curtain walls	Façade damage	Mid-span deflection	W_s or E_s	Span/250
Fixed glazing system	Glass damage	Deflection	W_s or E_s	$2 \times$ glass clearance

When considering wind actions of structural design, AS/NZS 1170.2:2002 (2002) gives specific explanations for tall buildings to evaluate the dynamic response factor in Section 6. Although the evaluation is divided into “along-wind response” and “cross-wind response” for tall buildings, various situations and factors are also considered. It is not hard to identify that the influence or contribution of non-structural components in resisting wind loading has not been considered. This is pointed out clearly and separately in the design requirements for high-rise buildings. In this case, future research may be required to identify the contribution of non-structural components and the interaction of structural and non-structural components in resisting wind loading.

Moreover, the serviceability criteria for tall buildings under wind load are given in Appendix G, AS/NZS 1170.2:2002 (2002). The methods given for calculating peak accelerations for both along and across winds use an average mass per unit height as one parameter. However, in this statement, it is still not very clear whether this average mass should include the non-structural components or not.

AS/NZS 1170.4:2007 (2007) is the standard of the earthquake in Australia and New Zealand. In this latest earthquake design standard, detailed requirements and design methods are provided for the non-structural components, in Section 8. In this section, the influence of connection properties on the damage of different non-structural components under earthquake load is discussed. Detailed calculations on forces generated to different non-structural components during earthquakes are also provided. These are all evidence that engineers in Australia have started taking account of the participation of non-structural components in structural performance.

According to AS/NZS 1170.4:2007 (2007), architectural, mechanical and electrical components and their fixings (the classification of response sensitivity of different types of non-structural components can refer to Table 2-2) should be designed to resist horizontal earthquake forces in accordance with established principles of structural dynamics or determined from the equation 2-1:

$$F_c = a_{floor} \left[I_c a_c / R_c \right] W_c \leq 0.5W_c \quad 2-1$$

It is worth mentioning that the key non-structural components focused by this study, façades and partition walls, are deformation-sensitive. That means the Equation 2-1 may be less applicable to these two types of components.

In summary, it has been recognised that although the serviceability design of structures does not involve the design of the strength or failure of components, it plays an important role in governing behaviour of structures, especially in the design of tall buildings. When the lateral performance of buildings starts to dominate, the serviceability of a high-rise building is of high importance. AS/NZS 1170 Series provides detailed guidelines for the design of structures. It points out the importance of the interaction and integration between structural and non-structural components. Relevant requirements are given. Although it is fairly practical, some points need to be further considered:

- The detailed design of non-structural components in resisting wind loads;

- The identification of interactions between structural and non-structural components;
- The proportion of contributions from structural and non-structural components in resisting lateral loads.

2.4.3 AS 3600:2001 (2001)

AS 3600:2001 is the standard for design of concrete structures in Australia and New Zealand. Detailed design requirements and explanations are given for structural components such as beams, columns, slabs, walls. Bending moments, shear force, axial forces and the combined actions are used to dictate the strength of the element. Reasonable consideration for reduction or enhancement of each factor in different situations is discussed thoroughly.

It is noted that although detailed design guidelines have been provided for the separation of different structural components in concrete structures, the interrelation and interactions among elements are not clearly defined in this standard. According to Kuang and Li (2005), interactions among structural components should not be neglected as they could have a significant influence. Consequently, the demand for further investigation regarding the evaluation of the interaction of structural components is required.

AS 3600 is primarily the design guideline for the concrete structures. Detailed approaches and criteria are given to the design of structural components. However, in terms of the non-structural components and connections between structural and non-structural components, more detailed instructions are required so that holistic design of structures can be conducted practical and accurate.

2.4.4 FEMA 356 / 2000

FEMA 356 is the Pre-standard and Commentary for the Seismic Rehabilitation of Buildings prepared by Federal Emergency Management Agency and American Society of Civil Engineers. Different procedures for structural analysis are developed according to different elements and performance requirements.

When explaining the acceptance criteria, it points out that,

“In a typical building, nearly all elements, including many non-structural components, will contribute to the building’s overall stiffness, mass, and damping, and consequently its response to earthquake ground motion. However, not all of these elements are critical to the ability of the structure to resist collapse when subjected to strong ground shaking...exterior claddings and interior partitions can add substantial initial stiffness to a structure, yet this stiffness is not typically considered in the design of new buildings...”

This statement indicates the contribution of non-structural components to the overall structural stiffness, and the difference existing between reality and structural design. It states that there are stiffness contributions from non-structural components. However, since the contribution is very small, it is normally not typically considered in the design. Different observations were revealed by Gad et al. (1998; 1999a; 1999b) and Su et al. (2005) that a significant contribution to the overall stiffness of the structure is made by non-structural components. These inconsistencies merit investigation.

This standard (FEMA 356/2000) identifies and proposes procedures for rehabilitating architectural, mechanical, and electrical components. To the force-sensitive components, default equations for seismic design forces on non-structural component are given as Equation 2-2 and 2-3:

$$F_p = 1.6S_{XS}I_pW_p \quad 2-2$$

$$F_{pv} = 2/3F_p \quad 2-3$$

When default equations do not apply, horizontal seismic design forces on non-structural components shall be determined in accordance with Equation 2-4:

$$F_p = \frac{0.4a_p S_{XS} I W_p \left(1 + \frac{2X}{h}\right)}{R_p} \quad 2-4$$

F_p calculated in accordance with Equation 2-4 is based on the stiffness of the component and ductility of its anchorage, but it need not exceed the default value of F_p calculated by Equation 2-2 and shall not be less than F_p computed in accordance with Equation 2-5:

$$F_p(\text{min}) = 0.3S_{XS}I_pW_p \quad 2-5$$

While the vertical seismic forces shall be determined in accordance with Equation 2-6 and 2-7:

$$F_{pv} = \frac{0.27a_p S_{XS} I_p W_p}{R_p} \quad 2-6$$

$$F_{pv}(\min) = 0.2S_{XS} I_p W_p \quad 2-7$$

For deformation sensitive components, for example façades and partition walls (in FEMA 356/2000, both force analysis and deformation analysis are required for these components), the standard (FEMA 356/2000) states that when non-structural components are anchored by connection points at different levels x and y on the same building or structural system, drift ratios (D_r) shall be calculated in accordance with Equation 2-8:

$$D_r = (\delta_{xA} - \delta_{yA}) / (U - u) \quad 2-8$$

When non-structural components are anchored by connection points on separate buildings or structural systems at the same level x , relative displacements (D_p) should be calculated in accordance with Equation 2-9

$$D_p = |\delta_{xA}| + |\delta_{xB}| \quad 2-9$$

Considering architectural components, the definition, acceptance criteria and evaluation requirements are identified in Table 2-4. From this table and previous sections, it is easy to identify how the evaluation and rehabilitation requirements for single structural and non-structural components are detailed. Further discussions on the influence of structural performance/evaluation requirements caused by interaction and integration of both non-structural and structural components are still forthcoming.

2.4.5 Eurocode 8 (1998)

Eurocode 8 sets out the design provisions for earthquake resistance of structures in Europe. It is part of the Eurocode 1-9 series and provides guideline for structural seismic actions, buildings, materials and individual elements, foundations and geotechnical aspects.

It is clearly pointed out in Part 1.1 that the costs of earthquake damage and the associated limitations of use of structures are “*disproportionately high in comparison with the cost of the structure itself*”. Thus, in Part 1.1 and 1.2, the following points are emphasized by the code:

“It shall be verified that under the design seismic action the behaviour of non-structural elements does not present risks to persons and does not have a detrimental effect on the response of the structural elements.” (Clause 4.4, part 1.1)

...

“Non-structural elements (appendages) of buildings (e.g. parapets, gables antennae, mechanical appendages and equipment, curtain walls, partitions, railings) that might, in case of failure, cause risks to persons or affect the building main structure or services of critical facilities, shall, together with their supports, be verified, to resist the design seismic action.” (Clause 3.5.1(1), part 1.2)

According to the code, seismic effects on non-structural elements can be determined by applying a horizontal force F_a determined from Equation 2-8:

$$F_a = (S_a \times W_a \times \gamma_a) / q_a \quad 2-8$$

The seismic coefficient S_a is defined in Equation 2-9:

$$S_a = \alpha \times 3 \times \left(1 + \frac{Z}{H}\right) / \left(1 + \left(1 - \frac{T_g}{T_1}\right)^2\right) \quad 2-9$$

When considering the serviceability limit state, the following limitations of inter-storey drift should be fulfilled:

For buildings having non-structural components made of brittle materials and are attached to the structure,

$$\frac{d_r}{v} \leq [0,004] \times h \quad 2-10$$

For buildings having non-structural components fixed in a way so as not to interfere with structural deformations,

$$\frac{d_r}{v} \leq [0,006] \times h \quad \mathbf{2-11}$$

In Part 1.3, concrete, steel, and masonry structures are discussed in detail respectively. When considering the resistance criteria for secondary components of concrete structures, the code points out that non-structural components may also contribute to energy dissipation, and appropriate measures should be taken against possible local adverse effects due to the interaction between structural and non-structural components. This indicates that the important role played by the interaction between structural and non-structural components has already been recognised in concrete structure design.

Table 2-4 Seismic evaluation criteria of architectural components

Category	Component Behaviour	Acceptance Criteria		Evaluation Requirements	
		Life Safety Non-structural Performance Level	Immediate Occupancy Non-structural Performance Level		
Exterior Wall Elements	Adhered Veneer	Deformation-sensitive	Backing shall be adequately anchored to resist seismic forces. The drift ratio shall be limited to 0.02	Backing shall be adequately attached to resist seismic design forces. The drift ratio shall be limited to 0.01	Shall be evaluated by visual observation and tapping to discern looseness or cracking
	Anchored Veneer	Acceleration-sensitive and Deformation-sensitive	Backing shall be adequately anchored to resist seismic forces. The drift ratio shall be limited to 0.02	Backing shall be adequately attached to resist seismic design forces. The drift ratio shall be limited to 0.01	Stone units shall have adequate stability, joint detailing, and maintenance to prevent moisture penetration from weather that could destroy the anchors. The anchors shall be visually inspected and tested to determine capacity if any signs of deterioration are visible
	Glass Block Units and Other Non-structural Masonry	Acceleration-sensitive and Deformation-sensitive	Shall be capable of resisting both in-plane and out-of-plane forces or shall meet the requirements of the prescriptive procedure if permitted. The drift ratio shall be limited to 0.02	Shall be capable of resisting both in-plane and out-of-plane forces or shall meet the requirements of the prescriptive procedure if permitted. The drift ratio shall be limited to 0.01	Shall be evaluated based on the criteria of section 2110 of IBC (2000)
	Prefabricated Panels	Acceleration-sensitive and Deformation-sensitive	Shall be capable of resisting both in-plane and out-of-plane forces or shall meet the requirements of the prescriptive procedure if permitted. The drift ratio shall be limited to 0.02	Shall be capable of resisting both in-plane and out-of-plane forces or shall meet the requirements of the prescriptive procedure if permitted. The drift ratio shall be limited to 0.01	Connections shall be visually inspected and tested to determine capacity if any signs of deterioration or displacement are visible
	Glazed Exterior Wall Systems	Acceleration-sensitive and Deformation-sensitive	Shall be adequately anchored to resist seismic forces. ⁽¹⁾	Shall be adequately anchored to resist seismic forces. ⁽²⁾	Shall be evaluated visually to determine glass type, and anchors.
	Partitions	Acceleration-sensitive and Deformation-sensitive	Non-structural heavy partitions shall be capable of resisting out-of-plane forces. The drift ratio shall be limited to 0.01 Non-structural light partitions need not be rehabilitated for this level	Non-structural heavy partitions shall be capable of resisting out-of-plane forces. The drift ratio shall be limited to 0.005 Non-structural light partitions shall be capable of resisting out-of-plane forces. The drift ratio shall be limited to 0.01	Shall be evaluated to ascertain the type of material

Category	Component Behaviour	Acceptance Criteria		Evaluation Requirements
		Life Safety Non-structural Performance Level	Immediate Occupancy Non-structural Performance Level	
Ceilings	Acceleration-sensitive and Deformation-sensitive	Where rehabilitation is required, ceilings in different categories shall be strengthened to resist seismic forces according to relative requirements and procedures	Ceilings shall be capable of resisting relative seismic forces and accommodating relative displacement according to their different categories.	The condition of the ceiling finish material, its attachment to the ceiling support system, the attachment and bracing of the ceiling support system to the structure, and the potential seismic impacts of other non-structural systems on the ceiling system shall be evaluated.
Parapets and Appendages	Acceleration-sensitive	Shall meet prescriptive requirements or shall be capable of resisting relative seismic forces	Shall meet prescriptive requirements or shall be capable of resisting relative seismic forces	The condition of mortar and masonry, connection to supports, type and stability of the supporting structure, and horizontal continuity of the parapet coping, shall be considered in the evaluation.
Canopies and Marquees	Acceleration-sensitive	Shall be capable of resisting both relevant horizontal and vertical seismic design forces	Shall be capable of resisting both relevant horizontal and vertical seismic design forces	Buckling in bracing, connection to supports, and type and stability of the supporting structure shall be considered in the evaluation.
Chimneys and Stacks	Acceleration-sensitive	Shall be capable of resisting relative seismic forces. Residential chimneys shall be permitted to meet the relevant perspective requirements	Shall be capable of resisting relative seismic forces. Residential chimneys shall be permitted to meet the relevant perspective requirements	The condition of the mortar and masonry, connection to adjacent structure, and type and stability of foundations shall be considered in the evaluation. Concrete shall be evaluated for spalling and exposed reinforcement. Steel shall be evaluated for corrosion.
Stairs and Stair Enclosures	Acceleration-sensitive or Deformation-sensitive	Shall be capable of resisting relative seismic design forces and accommodating the expected relative displacement.	Shall be capable of resisting relative seismic design forces and accommodating the expected relative displacement.	The materials and conditions of stair members and their connections to supports, and the types and stability of supporting and adjacent walls, windows, and other portions of the stair shaft system shall be considered in the evaluation.

Summarised from FEMA 356 (2000)

2.5 Review of Structure Measurement/Monitoring Techniques

Structure monitoring serves important purposes such as checking the as-built performance of structure against design criteria, identifying unusual loading conditions, or modifying the understanding of structural behaviour (Ogaja et al. 2001). To measure or monitor the lateral performance of structures, top deflection and modal behaviour of buildings under lateral loads are critical criteria. According to previous research (Celebi 2000; Ogaja 2000, 2001; Chan et al. 2006; Nickitopoulou et al. 2006; Aziz et al. 2006; Seco et al. 2007), to obtain the displacement and modal data, accelerometers and Global Positioning System (GPS) are the most popular measurement techniques used recently in the civil engineering measurement process.

2.5.1 Accelerometers

Accelerometers are widely used in various areas, especially civil, aerospace, and mechanical engineering, etc. for different measuring purposes (Allen et al. 1989; Bonato et al. 1997, 2000; Xiong et al. 1998; Lu and Law 2006; Fujii 2007; Mark and Reagor 2007; McGorry et al. 2007; Sahoo et al. 2007). More and more recognition has been given to accelerometers because of their substantive advantages and special features. Accelerometers have been widely used to facilitate analyses such as for time/frequency analysis, impact response measurement and analysis, structure damage detection, and aerospace development.

2.5.1.1 Features of Accelerometers

A piezoelectric accelerometer is an electromechanical transducer that generates an electrical output when subjected to vibration. The electrical output is directly proportional to the acceleration, over a limited frequency and dynamic range (Brüel and Kjær 1974).

In selecting accelerometers, the following basic features should be considered first.

- Sensitivity. *“The ratio of the accelerometer’s electrical output to the mechanical input is defined as the sensitivity of the accelerometer.”* (Brüel and Kjær 1974);
- Frequency range;

- Dynamic range. The dynamic range of an accelerometer is defined as the range over which the electrical output of the accelerometer. It will be directly proportional to the acceleration of its base;
- Operating temperature; and
- Self weight.

Ideally, an accelerometer with high sensitivity, maximum frequency range, minimum weight, and maximum operating temperature range will be the best choice. Unfortunately, the requirement of high sensitivity has direct conflict with the requirement of low self-weight and maximum frequency range. In this case, compromises are always made.

2.5.1.2 Advantages of Accelerometers

The main advantages of accelerometers can be summarised as follows.

- Accelerometers can be used for both low and high frequency measurement;
- Accelerometers are handy tools for a wide range of measurements, especially vibration measurement;
- Accelerometers can measure the high natural frequency of structures. According to Chan et al. (2006), an accelerometer can extract acceleration responses of structures with natural frequency up to 1000Hz.

In the structural engineering area, because of its flexibility and capability in structure measurement, the recording of acceleration responses of structures from accelerometers serves us well (Celebi 2000). Studies of such records have helped in assessing design and analysis procedures, improving code provisions, and correlating response with damage.

2.5.1.3 Disadvantages of Accelerometers

With the development of measurement techniques and the maturing of structural engineering, disadvantages of the accelerometer have been gradually exposed:

- Accelerometers are not efficient or effective in measuring the relative displacement of structures, the key parameter for assessing drift and stress conditions of structure (Celebi 2000);
- Double integration is required to obtain the displacement from an acceleration response and the results from integration may drift over time due to unknown integration constants (Chan 2006). Additionally, the level of accuracy of displacement calculated from accelerations has not been widely verified by observations;
- Accelerometers are insensitive to low frequency acceleration changes (Chan 2006);
- The measurement from an accelerometer can hardly accommodate a long time span;
- The influences of the surrounding environment such as temperature may generate significant errors in the result.

Because of the above constraints, data obtained from accelerometers appears to be insufficient in terms of the accuracy in the measurement of diverse structures, especially those with very low natural frequencies. Even though the accelerometers are still widely adopted in current academic and industrial applications, other measurement methods which can be integrated with accelerometers are required.

2.5.2 Global Positioning System (GPS)

GPS is a satellite-based navigation system which was developed by the U.S. Department of Defence originally for military usage. It was then quickly developed and made available to civilian users (El-Rabbany 2006). Nowadays, the applications of GPS cover numerous areas which include utilities industry, forestry and natural resources, precision farming, civil engineering, mining industry, retail industry, seafloor mapping, and so on. In civil engineering field, it has been widely involved in the real-time monitoring of structural deformation and frequency under certain loading conditions.

2.5.2.1 GPS segment

Basically, there are three segments in GPS: the space segment, the control segment, and the user segment (El-Rabbany 2006, Figure 2-7).

The space segment consists of a 24-satellite constellation. To ensure continuous worldwide coverage, four satellites are placed in each of six orbital planes. In this case, four to ten satellites will be visible from any position in the world. Each satellite transmits a signal, which contains two carrier frequencies, two digital codes, and a navigation message. Those signals are controlled by highly accurate atomic clocks onboard the satellite. The distance from user's receiver to the GPS satellite, navigation message, and other information can be all contained in the signal.

The control segment consists of a worldwide network of tracking stations to track the GPS satellites in order to determine and predict the satellite locations, system integrity, etc..

The user segment includes all military and civilian users. Users can receive the GPS signals with a GPS receiver and a GPS antenna. Those signals can be used to position a location anywhere in the world.

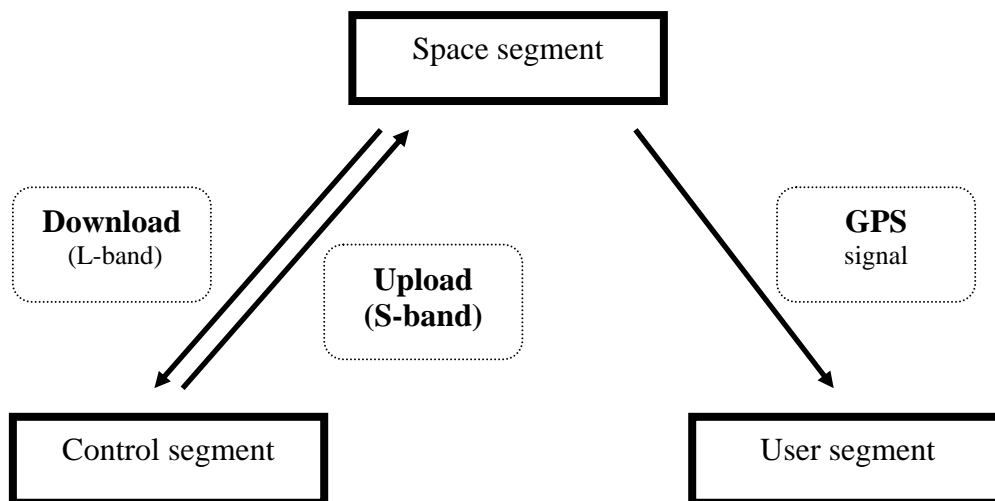


Figure 2-7 GPS segment (El-Rabbany 2006)

2.5.2.2 Advantages of GPS

GPS has revolutionised the surveying and navigation fields (El-Rabbany 2006). According to various researches, obvious advantages of GPS can be summarised as follows.

- GPS is a cost-effective tool for navigation and surveying, compared with other traditional methods. Around 50% of cost saving and more than 75% time saving can be achieved by using GPS (El-Rabbany 2006);

- GPS can measure directly position coordinates. Nowadays, the relative displacement of structure can be measured down to millimetres at rates of 10Hz or higher (Ogaja et al. 2001);
- GPS can be easily integrated with other conventional equipment;
- GPS can be widely applied to various areas such as land, marine, and structure monitoring, etc.;
- GPS can easily realise the real-time and long-term monitoring to structures;
- GPS can be a measurement method with relatively high accuracy.

In terms of structure monitoring,

- GPS is capable of detecting tall building response to extreme loading events such as windstorms with high velocities and earthquakes of average to high magnitudes in real-time (Ogaja et al. 2000);
- GPS is convenient in estimating the permanent displacements experienced by tall buildings (Ogaja et al. 2000);
- GPS is useful in detecting the deformation of structures (such as tall buildings) due to ground subsidence in a long-term manner (Celebi 2000; Ogaja 2000).

2.5.2.3 Disadvantages of GPS in Structure Monitoring

Although GPS has the remarkable advantages listed above, shortcomings are also inevitable since this technology is still under development. Focusing on the structure monitoring aspect,

- It is not so reliable to use GPS to collect data under low frequency. According to the research from Ogaja et al. (2000) on the Republic Plaza building, Singapore, the low frequency signal of tall buildings can not be easily recognised at 1Hz under normal loading conditions;
- It is not a cost-effective way of monitoring displacements of slow-deforming structures (Celebi 2000; El-Rabbany 2006). If GPS units can be used on structures, especially tall buildings, which are already instrumented with accelerometers, it will facilitate the comparison of absolute and relative displacements.

2.5.3 Current Observations

When comparing the advantages and disadvantages of both accelerometers and GPS, it is not hard to see that integrating GPS and accelerometers in the structure measuring and monitoring process will be a better solution than accelerometers-only or GPS-only methods (Celebi 2000; Ogaja 2000, 2001; Chan et al. 2006; Nickitopoulou et al. 2006; Aziz et al. 2006; Seco et al. 2007).

According to Celebi (2000), to monitor the dynamic response of structures, GPS units should be installed on buildings which have already been instrumented with accelerometers so that the comparison of absolute and relative displacements measured and calculated from different devices can be conducted. In Celebi's (2000) research, GPS was configured to obtain measurement data which indicated real-time average ratios and changes in dynamic characters of buildings. Under these circumstances, real-time information was achieved. The accurate time spot when a predetermined displacement threshold is reached can also be observed.

Nickitopoulou et al. (2006) analysed the accuracy and limitations of GPS in monitoring dynamic and quasi-static deformations of large flexible engineering structures. It is noted that GPS can monitor displacement with amplitude larger than 15mm, at a level of outliers less than 1.5%. Moreover, even if sometimes the displacement data obtained from GPS are noisy, high accuracy can still be achieved when computing the dominant frequencies based on GPS records. Differences between real and computed dominant frequencies were random in their study (Nickitopoulou et al. 2006), whilst all of them were smaller than 0.06Hz. Therefore, GPS has proved to be a promising measurement tool in civil engineering fields, acting complementarily to accelerometers.

An integrated GPS-accelerometer data processing technique was proposed by Chan et al. (2006). It adopted empirical mode decomposition and an adaptive filter to enhance measurement accuracy. This technique aimed to overcome the insensitivity of accelerometers under low-frequency and the multipath of GPS. The key task was to extract the mean displacement, the dynamic displacement and the total displacement of the structure from the combined GPS and accelerometer signals. The result verified the capability and efficiency of the new integrated system.

Results from the study conducted by Seco et al. (2007) illustrated that GPS allows for high degree of automation and it is reliable operating in diverse weather conditions.

However, difficulties in selecting installation locations exist for many studies. Moreover, errors of GPS are not random in nature.

Based on the information obtained from the review, it is concluded that in terms of the real building measurement, ensured by the complementary characteristics of GPS and accelerometers, an integrated GPS-accelerometer measurement system is the best solution for field measurement of a structure. By integrating these two devices, the reliability and accuracy of measurement data can be ensured. Also, the real-time and long-term structural monitoring can be achieved. However, given the constraints of both devices (GPS and accelerometers), the design of the integrated measurement system can be difficult. In laboratory tests, constrained by specific features of the structural model, such as the high frequency (much higher than the real building frequency), the limited installation areas, as well as the in-door environment, GPS may not be the suitable measurement equipment.

2.6 Review of Structure Modelling Techniques

Numerical analysis is the method used to implement and verify physical systems that engineers conceive (Bhat and Chakraverty 2004). Mathematical models and equation systems are employed as basic tools for the numerical method. Any problem originated from the real-world applications of algebra, geometry and calculus, as they have variables which varied continuously. During the past half-century, with the development of computer science, numerical solutions of engineering problems become simpler. Accuracy has also been improved by the development of systematic applications of algorithms and numerical techniques.

2.6.1 Characters of Numerical Analysis

Starting in ancient Egypt, numerical algorithms are about the same age as human civilisation. Follows are several key features of numerical analysis.

- Numerical linear and non-linear algebra. This refers to problems involving the solutions of systems of linear or non-linear equations, possibly with a very large number of variables;
- Approximation theory. This theory covers the approximation of functions and methods based on using such approximations;

- Solving differential and integral equations. Most mathematical models used in engineering are based on ordinary differential equations, partial differential equations and integral equations;
- The computer structure and properties affect the structure of numerical algorithms, especially when solving large linear systems.

In relation to the structural engineering applications, Katagiri et al. (2002) used spectral analyses and time-history analyses using motion-induced wind forces for high-rise buildings to discuss the validity of these two methods. Vibration equations were used to process the analyses. The corresponding results from the two analyses strongly indicated that the numerical analysis is very powerful to solve engineering problems. He and Song (2007) built their Computational Fluid Dynamics (CFD) model based on wind engineering theory. The results obtained from their research also added credits to the accuracy and flexibility of numerical analysis methods. Even though many studies did not directly involve numerical methods in their analyses, their structural models or programs were developed using different software based on numerical theories. This proves that numerical analysis methods are the foundation of most of the widely adopted modelling techniques.

Generally, numerical analysis methods have the following advantages:

- It defines the problem clearly. The numerical analysis method has sophisticated system which has been developed for many decades. It is supported by well developed theories in different scientific areas. It is a basic tool for solving many engineering related problems;
- It can be used flexibly in a wide range of areas. Various problems can be defined using different numerical models, within not only the engineering area, but also mathematical, physical, medical, and even social areas. Moreover, no matter what the scope of the problem is, numerical models can be developed in most situations.

2.6.2 Finite Element Analysis

“The finite element method is a numerical procedure that can be applied to obtain solutions to a variety of problems in engineering.” (Moaveni 1999)

From their origin which can be traced back to the early 1900s, modern finite element methods and theories experienced Courant, Boeing, Clough, Zienkiewicz and Cheung (Moaveni 1999). In 1971, a comprehensive general-purpose finite element computer program, ANSYS, was released for the first time.

Even though other computational tools involving the robustness criteria (Sarma and Adeli 2005; Wu et al. 2006), the energy dissipation models (Newland and Cebon 2002), and the evolutionary design tools (Kicinger et al. 2005), etc. provided great opportunity to the development of design optimization and the modelling technology in the structural engineering area, the finite element analysis is still the most popular analytical technique involved in structural analyses. With the development of various modelling software such as ANSYS, SAP, ABAQUS, etc., more and more structural problems are successfully solved using analytical models.

Observations of finite element modelling techniques may be obtained when different modelling procedures were used to analyse the performance of high-rise buildings under different loads (Pantelides et al. 1996; Mahendran and Moor 1999; Foutch and Yun 2002; Horr 2003; Lu and Chen 2005; Ravi Kumar et al. 2006; Wilkinson and Hiley 2006).

The influence of characteristics of elements. Foutch and Yun (2002) investigated 6 different models with different element dimensions to analyse the performance of steel moment frames under seismic loading. Components such as panel zones, columns and beams were defined and modelled separately. Because of the clear definition of elements, high accuracy and reliability were achieved in the results. It was found that the dimension of elements, the connection type, and the gravity of the modelled frame may affect the results significantly. Super-element methods were developed for finite element modelling processes as an efficient and effective way of modelling shear walls with and without openings (Li 2001; Kim and Lee 2003, Kim et al. 2005; Lu and Chen 2005). Under these circumstances, the characteristics of elements such as mass matrix and stiffness matrix become especially important.

The influence of model simplifications. Constrained by the development of computer science, two-dimensional (2-D) modelling was once popular for structure analyses. However, the insufficiency of 2-D models in representing the real structural performance has been generally noted by the researchers. Mahendran and Moor (1999) pointed out that 2-D models are not suitable for analysing the strength and the deflection

of steel portal frames. The comparison of the analytical results (both from the 2-D model and from the three-dimensional (3-D) model) and the full-scale test results showed that better match between analytical results and test results can be achieved by adopting the 3-D model. Thus, 3-D models should be widely encouraged in analyses of different structures to improve the accuracy and reliability of the analytical results.

The influence from features of different modelling software packages. According to Jan et al (2004), in order to compare and verify the reliability of results, different software packages/analysis models should be adopted when conducting structural analyses. They introduced three types of numerical simulation examples, NL-RHA, TLP and MPA to conduct the same pushover analysis for a high-rise building model respectively. The results vary from each other. This indicates that each software package or analysing programme has its own strength and weaknesses. In order to choose the most suitable modelling technique(s), detailed investigation and preliminary analysis is necessary.

The influence from verification procedures. Pantelides et al (1996) established two models for a one-story glass and aluminium shop-front wall system using ABAQUS and SAP 90 respectively. It was discovered that even if all the parameters adopted by the two models were the same, the results differed considerably. It indicates that the verification of the analytical results by other ways is necessary.

A great number of modelling methods have been developed to analyse the performance of high-rise buildings (Pekau et al. 1995, 1996; Ozturun et al. 1998; Mahendran and Moor 1999; Poulsen and Damkilde 2000; Kim et al. 2005).

- The “Finite Story Method” introduced by Pekau et al. (1995; 1996) can greatly reduce the unknowns of each storey in a high-rise building and thus improve the computational efficiency;
- The Program developed by Ozturun et al. (1998) has a special mesh generation subroutine and a graphic program for the finite element analysis of shear wall buildings. Beams or columns can be easily manipulated in this program. Therefore, the modelling process becomes more convenient.
- Mahendran and Moor (1999), Wilkinson and Hiley (2006) believed that the 2-D modelling analysis was not sophisticated enough to represent the real performance

of structures. Thus, a 3-D modelling method for the steel portal frame buildings is necessary.

- Poulsen and Damkilde (2000) provided details in how to consider the reinforcing steel bars and the tensile or compressive behaviour of concrete in limit states analyses when in-plane forces were applied to a reinforced concrete plate. This method is applicable to analyses of single reinforced element. However, when modelling high-rise structures, concerned with various constraints (such as the node number limitation in ANSYS), as well as the computational time, this method may only be used in substructure.
- Li (2001), Kim et al (2003; 2005), Lu and Chen (2005) presented an efficient example using the superelement method to model shear wall structures. Appropriate accuracy within reduced computational time was achieved by adopting this method. It was also noted that the dimension of elements, the connection type and the self-weight of the modelled frame could influence the results greatly (Li 2001; Foutch and Yun 2002; Kim and Lee 2003; Kim et al. 2005; Lu and Chen 2005).

In terms of software packages, results from research carried out by Pantelides et al (1996) and Jan et al (2004) indicated that variations of analytical results could be induced by adopting different software packages or analytical programmes even if the same structural model and same parameters were involved. Thus, a complicated selection process is required when choosing an analytical tool.

Considering some other foci;

- A great deal of modelling work focused on the behaviour of structures under seismic or wind loads (Zhang and Roschke 1999; Wang et al. 2001; Foutch and Yun 2002; Gu and Peng 2002; Hidalgo et al. 2002; Balendra et al. 2005; Campbell et al. 2005; Lin et al. 2005; Chan and Chui 2006; Wilkinson and Hiley 2006) since these two types of lateral loads are of the great concern in most countries and can cause severe damage to high-rise buildings.
- Most of the models were designed to conduct the ultimate limit state analyses or prediction. According to the studies reviewed in this chapter, researchers now can be very confident of the analyses and modelling of framed (Foutch and Yun 2002;

Wilkinson and Hiley 2006) and reinforced concrete shear wall structures (Hidalgo et al. 2002) under wind loading or seismic loading.

- It should be noted that most of analytical methods were based on 2-D models which introduced a lot of simplifications in the real performance along the third dimension. Even though some 3-D models were involved in the analysis, most of them were limited to model single elements.

In summary, constrained by current conditions, 3-D modelling analysis of high-rise buildings, especially the analysis facilitated by the full-scale building model with diverse non-structural components, is still a big challenge for current scholars and engineers.

2.7 Conclusions

This chapter provides a detailed discussion and review of high-rise building development, from both the material perspective and the structural perspective. Previous research on key factors which may influence the lateral behaviour of a tall building was investigated. Standards used both in Australia and abroad were reviewed in terms of the design consideration of non-structural components. Finally, the current structural measurement and modelling techniques were also reviewed.

It is noted from the review that the development of the structural forms and construction materials makes supertall buildings achievable. Steel and concrete are now the most widely adopted construction materials of tall buildings, supporting diverse types of structural forms, such as the framed structure, frame with cores, tube in tube structure, etc..

According to the review, it has been widely recognized that both the primary structure and the non-structural components contribute to the structural performance. However, even though various studies have been conducted to the tall building structures, few of them took the non-structural components into consideration. Even the design standards in many countries do not provide sufficient consideration to the integration of non-structural components into the structural analysis. Thus, a detailed study on this specific topic is highly demanded.

Based on the information collected from the literature review, a clear understanding of the structural performance of tall buildings as well as the current construction practice was achieved and it provided the foundation of this study. With the knowledge provided by previous research and literature, a detailed methodology for this study is developed. This research methodology is discussed in Chapter 3.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

In the previous chapter, it is identified that there is a gap existing between the real performance of tall buildings and the currently adopted structural analysis theories. That is, in the actual behaviour of buildings, the primary structure and the non-structural components are physically connected and they work together as a building system. However, in the current structural design and analysis, only structural components in the primary structure are designed as the load-bearing components whilst the non-structural components are considered to be detached to the main structure.

This study aims to analyse the structural performance based on evaluations of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis. To achieve this aim, a reliable method to accurately measure and analyse the global performance of tall buildings is necessary.

The methodology proposed for this study includes four main parts (as shown in the flow chart introduced in Chapter 1): the field reconnaissance, the preliminary finite element analysis, the laboratory testing, and the parametric study. These four parts not only follow a logical consequence but also interrelate to each other during the entire study.

This chapter discusses the methodology by using a work-break-down structure, levelling the activities and providing details for each activity involved in the study. The methodology developed for this study also considers the accuracy and reliability of the results obtained from each part of the research. Consequently, it provides a high-level confidence to the reliability of the conclusions that may be drawn from the study.

3.2 Overview of Holistic Method

To initialise this project, local industries were widely contacted and review of previous studies as well as the current practice was carried out. At the time the gaps between the

design practice and the real performance of tall buildings were identified, the project was initialised and the whole plan of study was ready to be triggered off.

Figure 1-1 in Chapter 1 illustrates the process of how to conduct this study by showing the flow chart of the holistic method proposed for the study. It is further developed in Figure 3-1 as a detailed self-explanatory chart to help break down the tasks in this study.

The four main parts are based on the logical and time sequences as first level/ primary tasks. They were conducted one after another, building up the framework of the study. Under each primary task, there are the second or the third level tasks depending on the scope of that part of activity. These sublevel tasks are parallel or ordinal, decided by the specific requirements of the primary tasks. For instance, after the initialisation of the project, to better define the scope of the study and to get a comprehensive understanding of the building performance, the first primary task of the study, proposed as the *Field Reconnaissance*, was conducted. It includes sublevel tasks such as the *building investigation*, the *detailed investigation of a case-study building*, the *discussion with building industry*, and the *review of previous studies*, etc.. Buildings at various locations including cities in Australia, mainland China and Taipei, were investigated. Base on the findings from the investigations, the scope of this study was refined and the in-depth understanding of tall building structures was obtained. At the completion of the *Field Reconnaissance*, gaps identified previously were validated. A Better understanding of building structures was obtained. The second, third and fourth primary tasks then follows one another in sequence, with the specific aims and objectives of their own, as shown in Figure 3-1.

The second primary task of this study is named *Preliminary Finite Element Analyses*. It was conducted to compare the structural performance with and without non-structural components. Design documentations and structure details such as properties of different connections, etc. of a case-study building were reviewed in order to obtain a better understanding of the mechanism of different types of connections and their applications. Based on the review of the design details of the case-study building, a solid foundation of the successive preliminary finite element analysis was built up. The typical non-structural components in this case-study building were modelled and integrated into the structural analyses. The influence of the connection properties were also analysed by introducing different types of constraints and boundary conditions of non-structural components. The preliminary results were then obtained. However, the reliability of the

conclusions drawn at this stage was required to be further validated by implementing laboratory tests.

The *Laboratory Testing* is another primary task of this study. A scaled model was developed according to the scaling theory adopted from Sabnis (1983). Different measuring sensors were selected and calibrated based on the measuring requirements of the laboratory model. This laboratory model was tested under different loading grades with different structural configurations. Meanwhile, finite element models were developed based on the laboratory model. The calibration of finite element models according to the testing results was conducted in order that further analysis and prediction can be carried out by using these finite element models. The laboratory testing was used to validate the parameters and preliminary conclusions drawn from previous activities. On completion of this activity, the contribution of different non-structural components was clarified.

An important activity shown in Figure 3-1 is *The Parametric Study*. It was carried out based on a high level confidence about the contributions of different non-structural components to the overall structural performance achieved in previous activities. The aim of conducting the parametric study was to precisely evaluate the influence of different non-structural components on the building performance. The damage level of non-structural components was also identified by this study.

Explanation of the contents in each of the four main parts (first level tasks) of this study is provided in the following sections in this chapter. Four individual chapters are also produced to depict the work conducted for each task in detail.

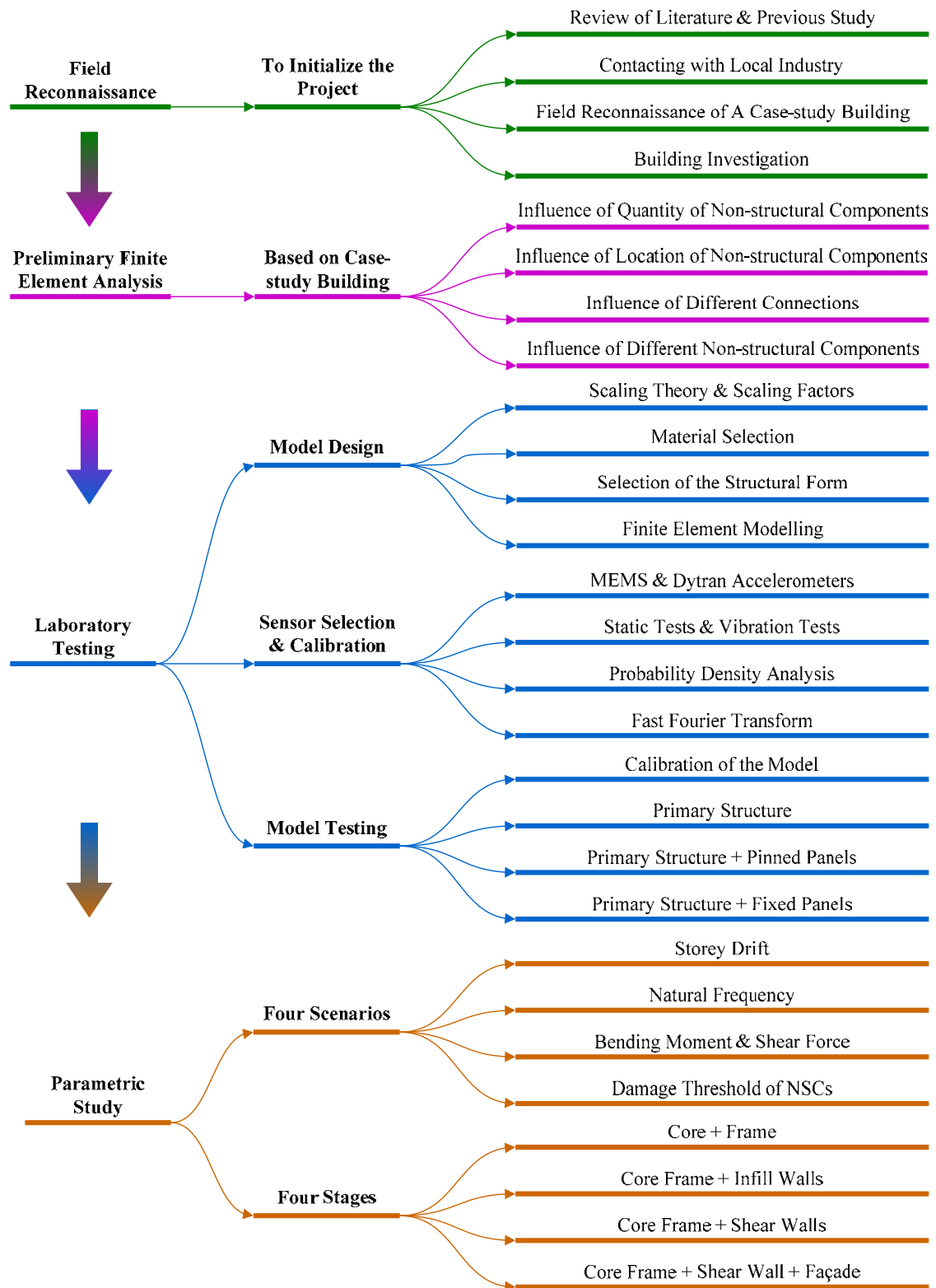


Figure 3-1 Holistic methodology of the research project

3.3 Field Reconnaissance

The aim of conducting this task was to validate the gap between current theoretical analyses and construction practice identified from the review of the literature, to gain a better understanding of the performance as well as the structural forms of different types of buildings at diverse locations, and to define/refine the scope of this study.

As introduced in Section 3.2, the field reconnaissance includes the investigation of different buildings, focusing on their structural forms, design features and the key non-structural components involved in the design. The investigation was conducted in Australia, Taiwan and mainland China. Fifteen buildings were investigated, from which a better understanding of the building performance and different design emphases in various locations were obtained.

Details of the planned investigation are listed in Table 3-1.

Table 3-1 Field reconnaissance details

Country/ Region	City	No. of Buildings	Building Name
Australia	Melbourne	1	Dock 5
	Sydney	1	World Trade Tower
	Gold Coast	1	Q1 Tower
Taiwan	Taipei	3	1. Taipei 101
			2. Xinyi District Commercial Building
			3. City Hall Subway Apartment
P. R. China	Beijing	3	1. China World Trade Centre-stage 3
			2. Fortune Plaza
			3. Jinguang Building
	Tianjin	3	1. The New Education Centre, Tianjin University
			2. Jiali Commercial Building
			3. Tanggu Apartment
Dalian	3	1. Hope Mansion	
		2. Xinghai Building	
		3. Ganjingzi District Apartment	

The key issues attracting the major attention in the investigation are the structural form, the overall design of the building system, including both the primary structure and non-

structural components, and the relationship between the design requirements and the local geological and meteorological conditions.

Relevant background information was collected before the investigation to better understand the structural forms of the target buildings. Moreover, to ensure the efficiency and effectiveness of the investigation, some local industry organisations were also contacted and the following questions were discussed in detail.

- What are design foci of buildings in this specific region?
- Whether or not some non-structural components are considered into the structural analysis (depending on the building) and if not, how the isolation of those non-structural components is achieved?
- What are your perspectives of integrating non-structural components into the structural analysis?
- What are the commonly recognised practices in this region when designing tall building structures?

By completing the building investigation, a clear view of building systems in diverse regions was obtained. Broadly adopted design practices based on different local conditions were also thoroughly understood. These in turn, provided guidelines for defining/ refining the scope of this study.

3.4 Analyses of a Case-study Building

3.4.1 Field Investigation and Detailed Review of the Design Documents

Apart from the building investigation conducted previously in different regions, to observe detailed design issues appearing in the tall building design in a local area, reconnaissance of a case-study building was pursued.

The Dock 5 building located in Dockland, Melbourne, Australia, was selected as the case-study building. The building developer was contacted prior to the field investigation to ensure the access to the construction site and the conductivity of the field work. According to the developer, this building was designed as a residential building with multi-level underground car parks and ground floor foyer. By the time the

field investigation was carried out, the building was still under construction. This to a great extent meant that some internal details of the building, such as the connection details, the sewerage, and the concrete pouring process could be observed.

There are several important aspects in the investigation of the case-study building. The first one is to review the design documents. By reviewing the design drawings, the structural form, floor layout and details of different elements could be obtained. Another aspect of this activity is to discuss, with the structural engineers of this building, the overall design process of the building. The implementation of design details should also be identified. This requires site visit and close observation of the building under construction.

Thorough understanding of structural details of this specific building was achieved via the above activities. Detailed design information such as properties of construction materials, properties of diverse connections, design considerations, and analysing tools/methods involved in the design phase of the building, was also carefully documented and discussed.

On the basis of the information obtained, a well informed preliminary finite element analysis based on this case-study building was developed for the next step. Detailed analysis of the structural performance with and without the inclusion of non-structural components was conducted.

3.4.2 Preliminary Finite Element Analyses

After the field reconnaissance, a comprehensive understanding of structural forms, load resisting mechanisms and design processes of tall buildings located in various regions was achieved. Detailed information of the specific building was also collected.

The preliminary finite element analyses were designed according to the case-study building introduced in Section 3.4.1. The aim of this activity is to analyse the lateral performance of the tall building with and without the inclusion of non-structural components to the structural analysis. Thus, information, such as detailed material properties, structural forms, floor plans, and various connection details, obtained from the case-study building was used as the input the finite element models, to consolidate the reliability of the analyses.

The analytical software chosen for the study was ANSYS because of its flexibility in defining different types of elements and different levels of degrees of freedom in each single node, as well as other advantages outlined in Chapter 2.

In the analyses, assumptions were made, key non-structural components were identified, and the scope and limitations of the analyses, as well as the consequent results, were also listed. Under the clear definition of the scope of the analyses, the following three scenarios were established:

- Influence of the quantity of non-structural components on the overall structural performance;
- Influence of the properties of connections between non-structural components and the primary structure on the overall structural performance;
- Influence of the location of non-structural components on the overall structural performance.

Under each scenario, the key non-structural components identified were evaluated in detail.

At the accomplishment of the preliminary finite element analyses, some results were obtained, accompanying the preliminary conclusions drawn from observations. However, even though the reliability of the finite results can be partly assured by the information obtained from the case-study building, unavoidable simplifications that were made in the analyses required further validation of the modelling and analysing methods to ensure a higher confidence.

3.5 Laboratory Testing

The experimental program in this project used proper testing systems on the laboratory model to measure the overall structure performance under serviceability loading conditions so that the contribution of non-structural elements to the overall structure performance could be identified and thus, the previous conclusions drawn from the preliminary finite element analysis could be further validated.

There are three main components in this activity: model design, sensor selection and calibration and model testing.

3.5.1 Design of the Laboratory Model

A 1:100 scaled laboratory model was designed according to the scaling theory from Sabnis (1983). To enhance the design solution, finite element models were developed to facilitate the selection of materials and structures. Furthermore, the following factors were also considered in designing the laboratory model:

- The space limit of the laboratory;
- Constraints from the measurement sensors;
- Constraints of the testing system.

3.5.2 Sensor Selection and Calibration

When the lab model was designed, the testing system was required to be established. Different types of sensors were tested and selected specifically for this laboratory model. Dytran accelerometers and various Micro-Electro-Mechanical-Systems (MEMS) sensors were evaluated to ensure that the most appropriate and reliable measurement system could be chosen in the end. Cables, voltmeter, power boxes, signal conditioners and computers are all indispensable accessories of the measurement system.

3.5.2.1 Sensor Details

Dytran accelerometers

Two types of accelerometers were evaluated. One is the Dytran 3191A, the other one is the Dytran 3192A. Table 3-2 lists the specifications of these two types of accelerometers.

From Table 3-2, it is clear that the Dytran 3191A model is more suitable for the real building tests because of its high capacity in frequency measurement and its reasonable sensitivity. However, in terms of the laboratory model tests, because of the lower self-weight and smaller volume, the Dytran 3192A becomes a better choice. Nevertheless, detailed selection and calibration of these two sensors, based on the features and requirements of the final laboratory model were still necessary.

MEMS sensors

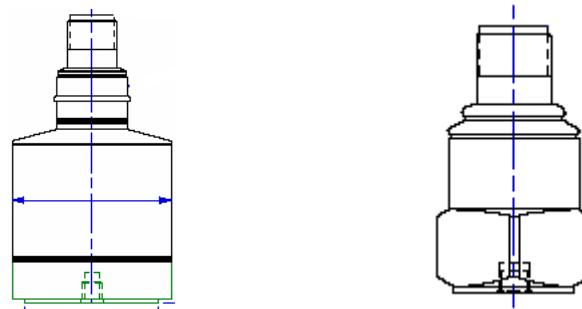
Over the past decade inertial sensor technologies have undergone a significant evolution with regards to their size, weight, power consumption and cost. What is still relatively undefined is the potential of these ‘new’ devices to augment GNSS performance. In this study, some of the MEMS sensors were tested in order to identify their reliability, accuracy and repeatability. On the basis of the calibration result, the decision was made on the application of the sensors in the test of the laboratory model.

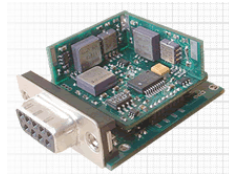
Four sensors were tested in this study. Details of the sensors are show in Figure 3-2 and Table 3-3.

Table 3-2 Comparison of specifications of Dytran 3191A and Dytran 3192A accelerometers (from the specification sheets)

Specifications	Dytran 3191A	Dytran 3192A
Weight	775 g	190 g
Size, Diameter x Height	2.00 × 3.65 inches	1.125 (Hex) × 2.25 inches
Material	Stainless steel	Stainless steel
Sensitivity	5.00 V/g	1.00 V/g
Frequency Range, +/- 5%	0.1 ~ 1000 Hz	0.5 ~ 1000 Hz

Drawing





#1 Crista_IMU



#2 Crossbow_TG



#3 X-Sens MTi



#4 Inertia Link

Figure3-2 MEMS sensors

Table 3-3 Summary of key features of MEMS sensors (from specification sheets)

No	Sensor	Measurement	Range	Error	Sampling Rate	Noise	Size	Weight
1	Crista_IMU	3- Axial Acceleration 3- Axial Gyros	$\pm 10g$ $\pm 300^\circ/sec$	$<1\%$ $<1\%$	$>1KHz$		2.05'' \times 1.55'' \times 1.00''	36.8g
2	Crossbow TG	3- Axial Acceleration	$\pm 2g$	$\pm 0.0085g$	$> 200Hz$	0.6mg rms	0.98'' \times 2.235'' \times 1.435''	110g
3	X-Sens Mti	3- Axial Acceleration 3- Axial Gyros	$<2g$ $\pm 300^\circ/sec$	$0.02 m/s^2$ $5^\circ/sec$	512Hz 120Hz	$0.001m/s^2/\sqrt{Hz}$ $0.1^\circ/sec/\sqrt{Hz}$	58mm \times 58mm \times 22mm	50g
4	Inertia Link	3- Axial Acceleration 3- Axial Gyros	$\pm 5g$ 360°	$\pm 0.005g$ $\pm 0.5^\circ (S)$ $\pm 2.0^\circ (D)$	1~250Hz		41mm \times 63mm \times 24mm	39g

3.5.2.2 Sensor Calibration

To select the most suitable sensors for the lab model testing, detailed sensor calibration was necessary. Figure 3-3 shows the planned arrangements for both static and vibration tests, to ensure the reliability of the sensors under diverse circumstances.

In the static tests, Sensors (No.1 to No. n) were attached to the test-bed which is a platform fixing to the structural wall of the laboratory. Relatively long measuring period (>24 hours) was required so that the reliability of the sensors for long-term measurement could be fully validated. Moreover, the tests were repeated three times in order to verify the repeatability of the sensors.

The static tests were conducted in a separate lab with restrictions on the access of people in order to reduce the external excitation/interruption and to simulate a static testing environment. However, vibrations of the building itself and some of the interruptions from night cleaning activities could not be avoided.

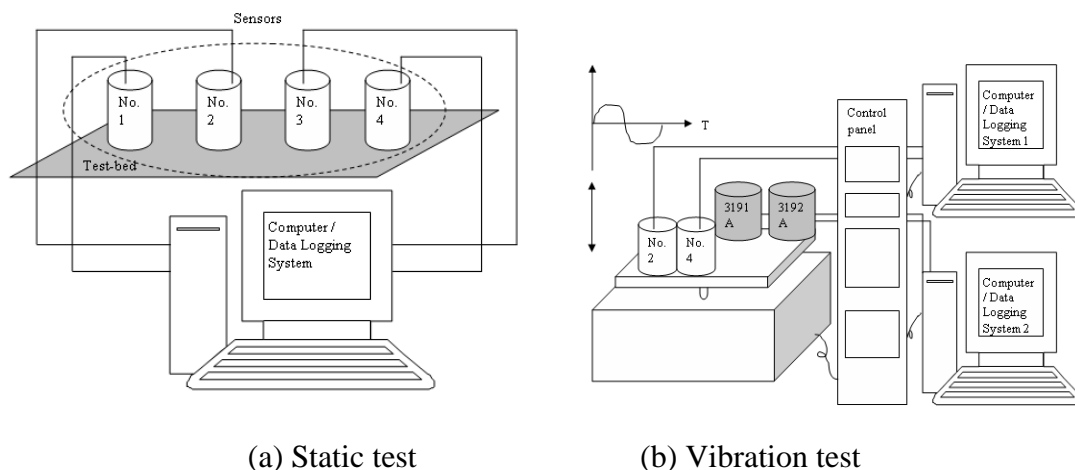


Figure 3-3 Arrangement of the static and vibration tests

The vibration tests were carried out in a structural laboratory using the Tinius loading machine which provided constant and controllable vibrations as inputs. During the vibration tests, two data logging systems were involved (Figure 3-4) because of the incompatibility of the two sets of data logging software. Since the Tinius loading machine was operated by hydraulic pressure from pre-stored mechanical oil, a certain level of instability of the machine performance should be expected.

The calibration results of the sensors are listed in Table 3-4. However, the operational and the analysing details are not provided in the thesis because of their low relevance to

the topic of this study. From the table it is concluded that MEMS sensors are suitable for the test which is not highly concentrating on the accuracy of the results and having high frequency motions (higher than 3Hz). In this study, the MEMS sensors are not satisfactory for the testing purpose.

Table 3-4 Summary of the performance of different sensors

No	Sensor	Measurement	Accuracy / Reliability		
			Static	Vibration (<3Hz)	Vibration (≥3Hz)
1	Crista_IMU	X- Axial Acceleration	?	--	--
		Y- Axial Acceleration	N	--	--
		Z-Axial Acceleration	Y	--	--
		X- Axial Acceleration	N	N	Y
2	Crossbow TG	Y- Axial Acceleration	N	N	Y
		Z-Axial Acceleration	N	N	Y
		X- Axial Acceleration	Y	--	--
		Y- Axial Acceleration	N	--	--
3	X-Sens Mti	Z-Axial Acceleration	Y	--	--
		X- Axial Acceleration	N	N	Y
		Y- Axial Acceleration	N	N	Y
		Z-Axial Acceleration	Y	N	Y

Notes:

1. "Y" represents yes, which means the sensor can reach its advertised functions
2. "N" represents no, which means the sensor can not reach its advertised functions
3. "--" means no comments
4. "?" means no conclusion

3.5.3 Laboratory Model Testing

The model testing was conducted after the laboratory model design and sensor selection. Influencing factors were required to be evaluated before the model tests, based on the final configuration of the lab model.

In this study, the model was tested under several grades of load with the following configurations:

- Primary structure;
- Primary structure with non-structural components fixed to it;

- Primary structure with non-structural components pin-connected to it.

Finite element models were also developed to verify the testing results. By analysing the results obtained from the lab model testing, conclusions on the contribution of non-structural components were drawn. Results obtained from the laboratory tests and the preliminary finite element analyses were compared and analysed in order to assure the reliability of the overall study.

3.6 The Analysis of Tall Buildings

On the basis of the information and conclusions obtained from the previous activities, (field reconnaissance, preliminary finite element analysis and laboratory model tests), the analysis of tall buildings which includes both theoretical analyses and a parametric study was carried out to evaluate the contribution of different non-structural components on the building performance, as well as the damage level of the individual component.

ANSYS was adopted as the analytical tool to analyse the structural characteristics with and without the inclusion of non-structural components, for the same reasons identified in Section 3.4. 3-D finite element models were developed to facilitate the analyses.

Prior to the parametric study, theoretical analyses focusing on individual structural forms (i.e. structural frame, frame with wall structure, frame with infill wall structure, structure with outrigger system) were conducted to enhance the reliability of the results achieved from the parametric study.

The parametric study was carried out by using the finite element models developed based on a typical steel-framed tall building with concrete service cores. There are four stages included in the analyses:

- Analysis of the primary structure (steel frame with concrete core);
- Analysis of the primary structure with infill walls;
- Analysis of the primary structure with shear walls;
- Analysis of the primary structure with shear walls and façade panels.

At each stage, the storey drift, natural frequency, bending moment and shear force distributions in structural elements (i.e. columns) were investigated. Moreover, the damage level of non-structural components under the allowable serviceability movements defined in the Australian design standards was also analysed.

3.6.1 Theoretical Analyses

To ensure the maximum reliability of the results obtained from various activities, it was necessary to conduct the theoretical analyses under different structural configurations. In this study, the following structures were analysed by using current structural analysis theories:

- Rigid frame;
- Frame with infill walls;
- Frame with shear walls;
- Shear wall with openings;
- Shear wall with façades;

To better validate the results from the parametric study, a series of appropriate finite element models were developed. Moreover, the material and structural details were required to be consistent in both analyses. In this case, the two sets of results obtained from both the theoretical analyses and finite element analyses were used to validate each other and to promote the confidence in the study.

The key issue of implementing the comparison of theoretical analyses and finite element analyses was to validate the theories adopted as well as the finite element modelling techniques. As long as the reliability of the theories was ensured, or reasonable interpretations of the theories were addressed to the analyses, the results from the parametric study could be ensured by using these theories.

3.6.2 The Parametric Study

The parametric study was to quantitatively evaluate the influence of different non-structural components on the overall building performance. The main non-structural components analysed were infill walls and façades. Different structural configurations

were proposed based on a “primary structure +” as shown in Figure 3-4. It was assumed that different non-structural components were attached to the primary structure to form different configurations such as “primary structure + infill walls”, “primary structure + façade”, etc. Thus the contributions of different non-structural components to the building performance could be clearly evaluated and shown in the analyses and the load/stress distribution and damage level of non-structural components could also be quantified.

From the parametric study, contributions of different non-structural components to the structural performance, such as structural stiffness, fundamental frequency of the structure and load distribution, were evaluated. Based on this, some preliminary design recommendations were proposed.

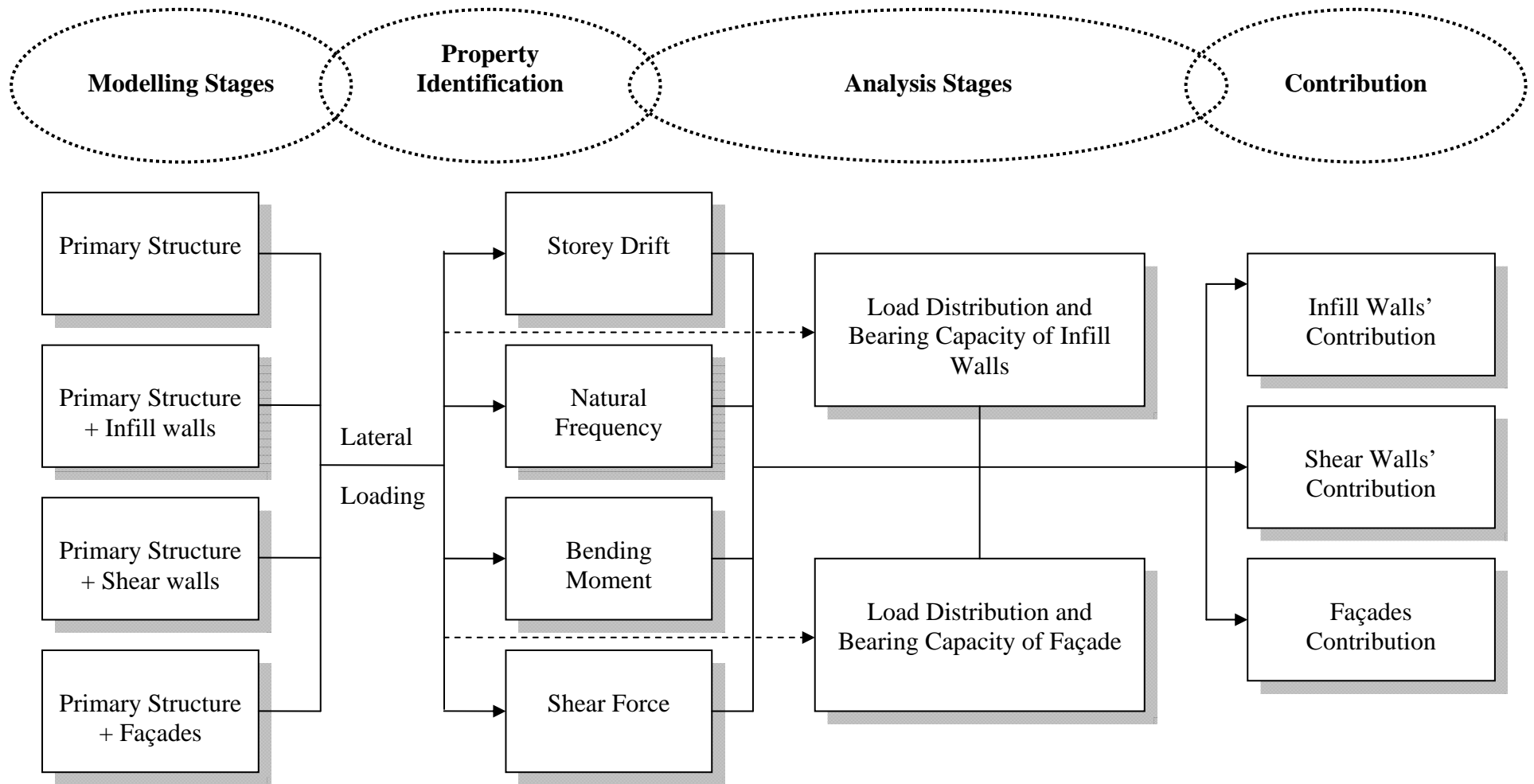


Figure 3-4 Procedures of the parametric study

3.7 Summary

The aim of this study is to analyse the structural performance based on evaluations of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis.

To achieve this aim, a reliable analytical method was required to be developed, in order that the influence of different non-structural components on the overall structural performance could be evaluated, and the damage level of different non-structural components could be assessed.

The methodology proposed for this study followed a logical sequence. The four main steps included were field reconnaissance, preliminary finite element analysis, laboratory test and parametric study. As explained in previous sections, the scope of the study was clarified through these four steps, which was accompanied by a build-up understanding of the building performance and by identifying the roles played by non-structural components in the overall building system.

Details of each of these steps of the study are discussed in Chapters 4, 5, 6 and 7.

CHAPTER 4

FIELD RECONNAISSANCE

4.1 Introduction

In order to better understand the global performance of tall building structures of how non-structural components are integrated into tall buildings, a field reconnaissance was conducted in various locations.

In this chapter, the details of the investigation of typical buildings in Asian-Pacific Region are discussed, followed by the comparison of the design focus and some concluding remarks obtained from the investigation. Through the field reconnaissance, a thorough understanding of design features and performance of buildings in different regions was obtained, based on which the gap between practice and design analysis is identified and the scope of this study further defined.

Fifteen buildings were investigated within the Asian-Pacific Region including Australia, Taiwan and mainland China. Issues such as structural form, typical design features, non-structural components and the design consideration were addressed in relation to the local geological conditions and the surrounding environment. An in-depth understanding of tall building design in different locations, as well as the performance of overall building system, was obtained from the investigation. It is also noted that local constraints influence the structural expressions of these tall buildings greatly. So does the formation of non-structural components.

Communication with local industries in the different countries greatly facilitated the understanding of current design focus of tall buildings in various locations. Design and construction companies Bovis Lend Lease Pty. Ltd. and Arup (Melbourne office and Beijing Office) were contacted during the field reconnaissance. Detailed discussions from the design perspectives of tall buildings relating to the integrated building system were conducted. From the communications, it is confirmed that in the design practice, non-structural components are seldom considered in the structural design, neither are they included in the advanced design analyses.

4.2 Investigation of Different Buildings

Different countries have different design standards for buildings according to their own geographical and geological conditions as well as the local environment. Moreover, the way of approaching buildings varies from culture to culture.

Due to rapid development of the economy and high density of the city populations, high-rise structures have become more and more popular in these three areas. Hundreds of magnificent tall buildings denote the skyline of cities. Nevertheless, threatened by different levels of earthquakes and/ or high gust winds, the focus of tall building design in these three countries is totally different. In this study, in-depth investigations were conducted focusing on the design features of tall buildings in each country. Comments and summary on the tall building design and performance in relation to the integration of different components, both structural and non-structural, are provided.

4.2.1 Aim and Objectives of the Building Investigation

The aim of this investigation is to thoroughly understand the performance of tall buildings and the load resisting mechanism of the tall building structure by comparing the differences existing in the design of tall building structures in different regions.

To achieve the aim, the following objectives should be met:

- Observe buildings chosen in different regions;
- Identify main design features of the buildings investigated;
- Identify main non-structural components of each building;
- Identify the connections between the non-structural components and the primary structure;
- Contact with the local engineers to understand the design focus of tall buildings in different locations.

Given consideration of the scope and limitations of this study (Chapter 1), 15 buildings were selected as typical samples representing different building designs in particular countries. Table 4-1 is a summary of buildings and regions visited.

Table 4-1 Building information for the field reconnaissance

Country/ Region	City	No. of Buildings	Building Name
Australia	Melbourne	1	1. Dock 5
	Sydney	1	2. World Trade Tower
	Gold Coast	1	3. Q1 Tower
Taiwan	Taipei	3	4. Taipei 101
			5. Xinyi District Commercial Building
			6. City Hall Subway Apartment
P. R. China	Beijing	3	7. China World Trade Centre-stage 3
			8. Fortune Plaza
			9. Jinguang Building
	Tianjin	3	10. The New Education Centre, Tianjin University
			11. Jiali Commercial Building
			12. Tanggu Apartment
Dalian	3	13. Hope Mansion	
		14. Xinghai Building	
		15. Ganjingzi District Apartment	

4.2.2 Brief Overview of the Environment and Geological Conditions

Local environments will have significant influence on the design focus of structures. To thoroughly understand the design features of these buildings, it is important to understand familiarize some background information of the environment of every country before the field reconnaissance.

Because of the distinct variations existing in the geological conditions and the environments, differences should be expected in the design of tall buildings in different areas.

This section provides a brief review of the environment and geological conditions of every region included in this field reconnaissance in order to facilitate the understanding of the different design features of these regions.

4.2.2.1 Cities in Australia

Melbourne, Sydney and Gold Coast are the three cities on which the reconnaissance was based in this study. Owing to their similar locations, climate and threats of natural hazards, these three cities are discussed together.

Australia is recognised as a mega-diverse country in terms of its climate and environment. Even though a large proportion of the land in Australia is semi-arid or desert, the major cities and its population are mainly located along the south-eastern and south-western coastlines (<http://www.bom.gov.au/lam/climate/>). Figure 4-1 is the climate map of Australia. From the map it can be seen that the climate of Australia is significantly influenced by the surrounding oceans. Except for the wide area of desert and grassland in the middle of Australia, in the major cities hosting most of the populations, the climate varies from temperate along the south-eastern coastline to subtropical on south-western coast and tropical and equatorial in the north.

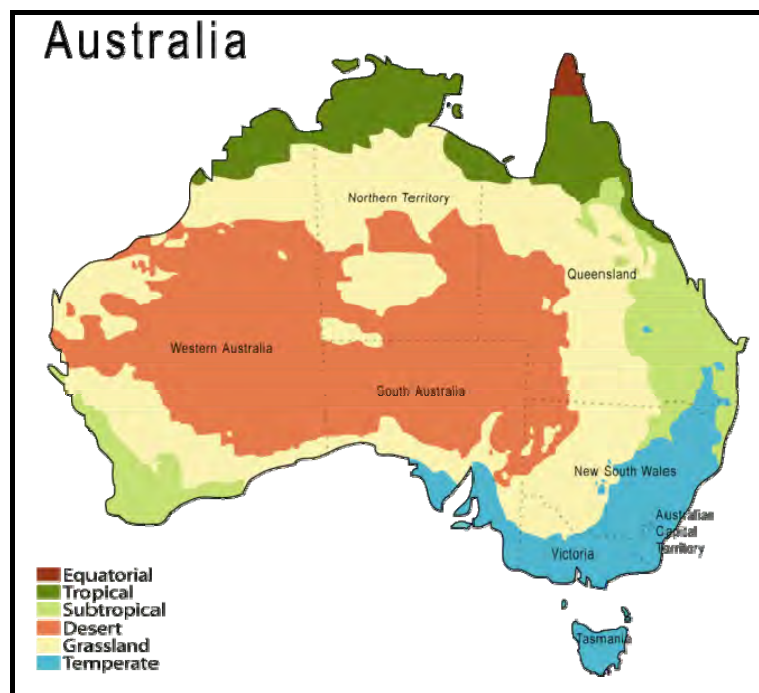


Figure 4-1 Climate map of Australia (<http://australia101.com/australia/climate-in-australia/>)

Some natural hazards including bushfires, cyclones, earthquakes, floods, landslides, severe weather, tsunamis, and volcanoes, impact on every Australian State and Territory (<http://www.australia.gov.au/>). However, the likelihood and consequence of each natural hazard vary from place to place. A scientific method of evaluating these natural hazards in each city or state in Australia has been well developed. Considerations of the consequences brought on by natural hazards for different structures should be assessed by judging the likelihood of hazards in the specific locations during the structural design life. Nevertheless, detailed introduction to the hazard quantification will not be provided since it is beyond the scope of this study. Generally, cyclones are severe in the northern

part of the country and only a small area in the south-western part of Australia (near Perth) has potential high seismic hazard level (Figure 4-2). In the cities discussed in this study (Melbourne, Sydney and Gold Coast), even though these two hazards are both rare, they should not be ignored in the design of structures. Thus, in the design of tall buildings in these three cities, wind load always governs the lateral stiffness whilst the earthquake or cyclone design still needs to be carefully considered.

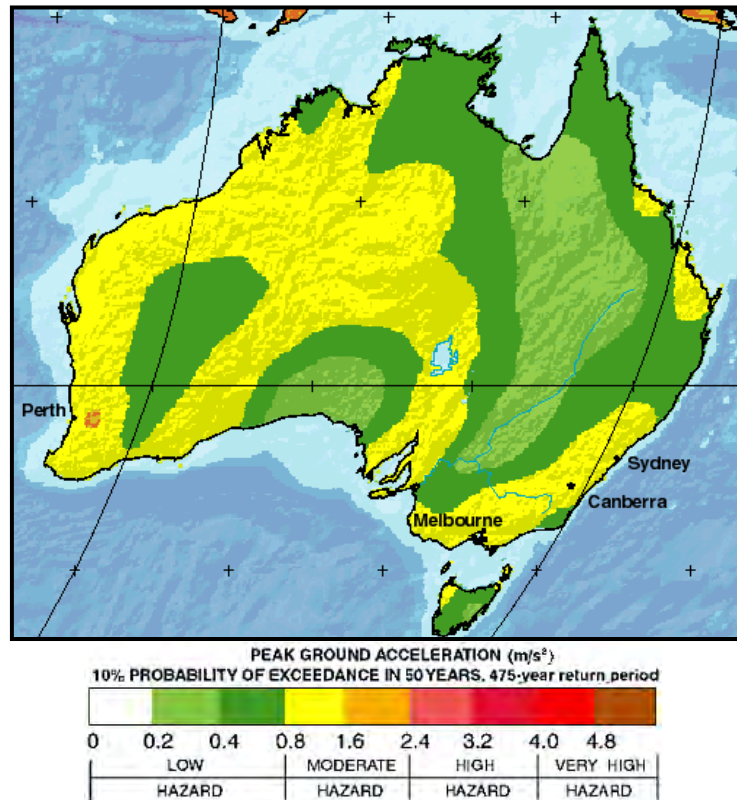


Figure 4-2 Seismic hazard map of Australia

(<http://geology.about.com/library/bl/maps/blaustraliaseisemap.htm>)

4.2.2.2 Taipei, Taiwan

Taipei, the capital city of Taiwan, is a small island surrounded by the East China Sea, South China Sea, and Philippine Sea. It is an island located in a complex tectonic area between the Eurasian Plate and the Philippine Plate (The Republic of China Yearbook 2008). The Taipei basin is situated on soft sandy soil sediments with high ground water table. From the East-Asian Seismic Map (Figure 4-3), it is not hard to identify that the peak ground acceleration in Taiwan is higher than $4.8m/s^2$, which in descriptive terms represents “Very High Hazard”. Meanwhile, because of the seas, Taiwan’s climate is marine tropical. Typhoons are a common visitor to this island. The northern part, including Taipei, has a long rainy season from January to March. Moreover, the whole

island is dominated by hot and humid weather from July to September. Thus, regarding the tall building design, building performance in earthquakes and design of structures under typhoons are the main issues need to be considered.

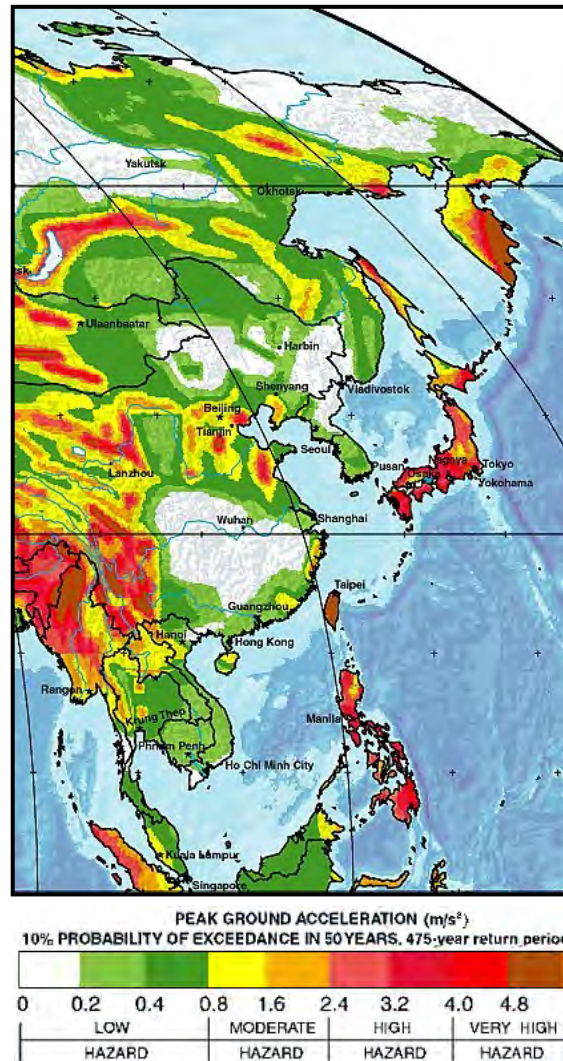


Figure 4-3 East-Asian seismic map

(<http://geology.about.com/library/bl/maps/blaustraliaseismap.htm>)

4.2.2.3 Cities in P.R. China

Because of the diversity of the local environment in the three cities investigated in P.R. China, separate introductions are provided for each city in this section relating to the local environment so as to better address the possible structural design considerations in different cities.

Beijing, P.R. China

Beijing, the capital city of People's Republic of China, is an inland city lying in the northern part of China (<http://www.britannica.com/>). In terms of the geography, climate, and natural environment, Beijing is a city sitting “at the northern tip of the triangular North China Plain” (MacKerras and Yorke 1991). It is shielded by mountains to the north, northwest, and west. Thus Beijing's climate is monsoon-influenced humid continental climate, which means humid and hot in summer whilst dry, windy, and cold in winter. Moreover, because of the erosion of the desert in northern and north-western part of China, dust storm happens seasonally in Beijing. The East-Asian Seismic Map (Figure 4-3) shows that the hazard level in most of Beijing is “moderate”, with the predicted peak ground acceleration of $0.8\sim 2.4\text{ m/s}^2$. However, some eastern areas of Beijing are categorised into areas with potential “high to very high” seismic hazard which have peak ground acceleration of $2.4\sim 4.0\text{ m/s}^2$. The geotechnical condition of Beijing is rather complicated, because of the frequent ground movements in ancient eras. However, the construction site within the CBD area, where the investigated buildings are located, has different types of solid rocks, boulders and cobbles composing its ground, providing good site conditions for construction.

From the structural design perspective, it is not hard to identify that both the seismic load and wind load should be seriously considered in the design of tall buildings.



Figure 4-4 China political map (<http://www.chinamapxl.com/political-map.html>)

Tianjin, P.R. China

Tianjin is one of the four municipalities that have provincial-level status (the other three cities are Beijing, Shanghai, and Chongqing). It is also the third largest city, ranked only

after Shanghai and Beijing (<http://www.tj.gov.cn/english>). The climate and seismic hazard levels in Tianjin are similar to that in Beijing, for they are closely located to each other (Figure 4-4). However, great differences exist in the geological conditions of these two cities. Beijing has rock (granite) underneath the ground in most areas whilst Tianjin typically has soft soil underground conditions. This can significantly influence the design and construction of structures. Even though analysis of foundation is not within the scope of this study, the difference of the underground conditions of these two cities will directly lead to the variation in the design of buildings and thus variations of structural expressions even using the same design code.

Dalian

Dalian is China's northernmost ice-free seaport (<http://www.dl.gov.cn/english>). The city also has a monsoon-influenced humid continental climate, similar to that in most parts of China. The Seismic map of Eastern Asia (Figure 4-3), predicts Dalian as a city with potentially moderate seismic hazard. The construction sites of Dalian city mainly have solid rocks with very limited groundwater, forming better ground conditions than Tianjin, in terms of construction.

4.2.3 Building Investigation

Based on information about the local environments and geological conditions described in previous sections, detailed investigations were carried out for the fifteen buildings listed in Table 4-1 in Section 4.2.1.

In this section, detailed discussions on individual buildings as well as the comparison and summary of design features of these buildings is developed, on the basis of observations and analyses of the selected buildings together with communications with local industries.

4.2.3.1 Buildings in Australia

Three buildings located in three different cities in Australia were investigated: the Dock 5 building located in Docklands, Melbourne, Victoria; the World Tower in Sydney, New South Wales; and the Q1 Tower in the Gold Coast, Queensland. As discussed in Section 4.2.2, the three cities have similar hazard exposure which can be expressed as a very low likelihood of earthquakes and cyclones. Under this circumstance,

considerations in the design of buildings in these cities tend to be similar, even though it may differ slightly from part to part.

Dock 5 in Melbourne, Victoria

Dock 5 is developed by Bovis Lend Lease Pty. Ltd. as the first residential building in the redevelopment of Docklands, Melbourne. The architects of this building are John Wardle Architects and HASSELL - Architects in Association. Its structural consultants are Arup and Connel Wagner. In 2008, Dock 5 won the RAIA Best Overend Award for Residential Architecture - Multiple Housing (VIC).

In terms of structural features, the building is located along the eastern seaboard of Melbourne, composing of 32 storeys. The main building is reinforced concrete structure, with a concrete core and two sets of shear walls integrated by floor slabs. The floor plans of Dock 5 are very complicated and vary throughout the building height. Figure 4-5 (a) and (b) shows the building in-use and under construction respectively.



(a) Dock 5 in-use

(b) Dock 5 under construction

Figure 4-5 Dock 5, Dockland Melbourne, VIC

It was confirmed by the structural engineer (Arup Melbourne Office) that due to its coastal location as well as the weather condition in Melbourne, wind load governed the overall design of the lateral resisting system of the building.

The key non-structural components identified in this building are partition walls and glass façades. Based on the Australian Standards and the discussion with the structural engineers, these non-structural components are considered to be isolated from the structural design and are not incorporated in the structural system.

Detailed investigation on this building was also conducted. The findings and results from preliminary finite element analysis will be discussed in Chapter 5.

World Tower, Sydney, New South Wales

The world tower is located in Liverpool Street, Sydney, New South Wales. It is a 230m high skyscraper, having 73 above ground level and 10 underground basement levels as shown in Figure 4-6. This building was constructed by Meriton Apartments Pty Ltd, (owned by developer Harry Triguboff) and it was the 2004 Bronze recipient of the Emporis Skyscraper Award. The World Tower was once the tallest residential building in Australia. The architect of this building is Nation Fender Katsalidis and the structural engineer of the World Tower is Connell Wagner, Sydney.

High-strength concrete was used in the construction of this building. The lateral resisting system of this building mainly includes (Dean et al. 2001): (a) a central core of reinforced concrete shear wall elements; (b) the perimeter “superframe” of columns, and belt beams located on every third floor; (c) two pairs of 8-storey high triangulated post-tensioned outriggers between core and perimeter columns centred about both mid-height plant levels.

In terms of the design loads, wind load was assessed as the dominant lateral load in the east-west direction whilst earthquake load was determined as the governing lateral force along the orthogonal direction (Dean et al. 2001) based on detailed computation taking into account the local environment and the geological conditions. Wind tunnel testing of this building was conducted by MEL Consultant at Monash University and predictions of the building behaviour such as fundamental frequency and maximum deflection under wind load was made through the wind tunnel analysis.

The main non-structural components identified in this building include façades (appears as curtain walls), partition walls, and stairs. Based on the current observations and a review of the design standards, incorporating some points in published research papers using this building as the case-study building (Dean et al. 2001), the non-structural components were separately designed according to their own design standards (e.g. the design of façade system). However, non-structural components were excluded in the structural design of the building. Further, none of the evidence mentioned in the previous discussion on structural features of this building (e.g. the wind tunnel testing and the design of the lateral resisting system, etc.) show any consideration of the integration of non-structural components in the overall structural design.



Figure 4-6 World Tower, Sydney, NSW

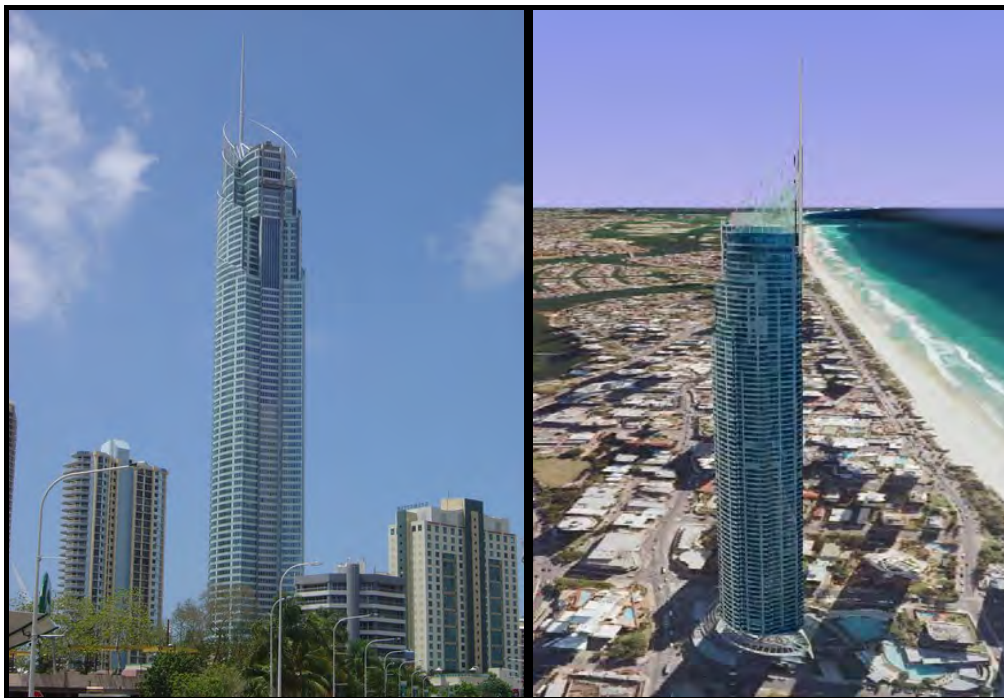
Q1 Tower, Gold Coast, Queensland

The Q1 Tower (Queensland Number One) is located in Surfer's Paradise, Gold Coast, Queensland. It is a super tall skyscraper, having 78 storeys with a roof height of 275m. However, including the top spire/antenna, the total height of this building comes to 323m, which makes it the tallest residential building in Australia.

Q1 Tower was developed by The Sunland Group and built by Sunland Constructions. The architect of this building was Atelier SDG. The building was the Silver Award winner of the 2005 Emporis Skyscraper Award.

The building is supported by 26 piles, having two metres in diameter that extend 40m into the ground then up to four metres into solid rock. The Q1 Tower has the Australia's only beachside observation deck: QDeck, which is 230m above the sea level. This building is designed in an oval shape inspired by the Sydney Opera House and the 2000 Sydney Olympic torch. Apart from its unique shape, Q1 Tower is a typical concrete core with bundled perimeter columns structure. The major construction material is reinforced concrete. Large amount of glass panels are also used in the construction of glass curtain walls and façade.

Figure 4-7(a) shows the street view of the Q1 Tower. A 3D model of the tower is also obtained from Google Science (Figure 4-7(b))



(a) Street view

(b) 3-D model view

Figure 4-7 Q1 Tower, Surfers Paradise, Gold Coast, QLD

(<http://sketchup.google.com/3dwarehouse/>)

4.2.3.2 Buildings in Taipei, Taiwan

There are total 3 buildings that were investigated in Taipei: Taipei 101 building, the City Hall Subway Apartment Building and the Xinyi District Commercial Building.

Taipei 101 Building

According to Taiwan Yearbook (2008), Taipei 101 is a *landmark skyscraper* in Xinyi District, Taipei. This 101-storey building is designed by C.Y. Lee & Partners and constructed primarily by KTRT Joint Venture. At the time of writing, Taipei 101 still officially held the title of “the world tallest building” authorized by CTBUH, the arbiter of tall building height. The building also received the Emporis Skyscraper Award in 2004, and was hailed as one of the Seven New Wonders of the World (Newsweek magazine 2006) and Seven Wonders of Engineering (Discovery Channel 2005). Upon its completion, Taipei 101 claimed its official records for:

- *Ground to highest architectural structure (spire): 509.2m;*
- *Ground to roof: 449.2m;*
- *Ground to highest occupied floor: 439.2m;*
- *Fastest ascending elevator speed: 16.83m/s (60.6 km/h);*
- *Largest countdown clock: on display every New Year's Eve;*
- *Tallest sundial.*

Figure 4-8 shows the view of Taipei 101 building from different directions. In terms of the structural features, Taipei 101 uses high-performance steel construction. Massive columns and enhanced bracing systems were adopted to achieve both the rigidity and the flexibility aimed to resist typhoon and earthquake loads. There are 36 columns supporting Taipei 101, including 8 “mega concrete” columns. Outrigger belt-trusses connect the columns in the building's core to those on the exterior every eight-storey. This building is a mega-frame structural system with a central braced core connected to perimeter columns on each building face, the total dead and live loads at every floor are transferred to the sloping exterior columns, thereby the structural capacity to withstand lateral loading is enhanced (Fan et al. 2009). To control the storey drift and vibration caused by lateral loads such as high wind and earthquakes and to stabilise the building

against the excessive movement, a 660-ton tuned mass damper has been installed inside the building on the top levels (Figure 4-9).

In terms of the non-structural components, along the surface of the building, double glazed glass curtain walls are used for heat and UV protection. The impact bearing limit of the glass is 7 tons. Inside the building, some partition walls can also be identified whilst most of the areas are open due to its multi-purpose usage including retail malls, observation storeys, and private clubs.

Analyses and discussions on the seismic performance and the structural system of the Taipei 101 building were provided in some research (Gunel and Ilgin 2007; Fan et al. 2009). However, there is not much information showing that any consideration was given to the integration of non-structural components into the structural analysis in the design of the Taipei 101 building.



Figure 4-8 Taipei 101, Taipei, Taiwan

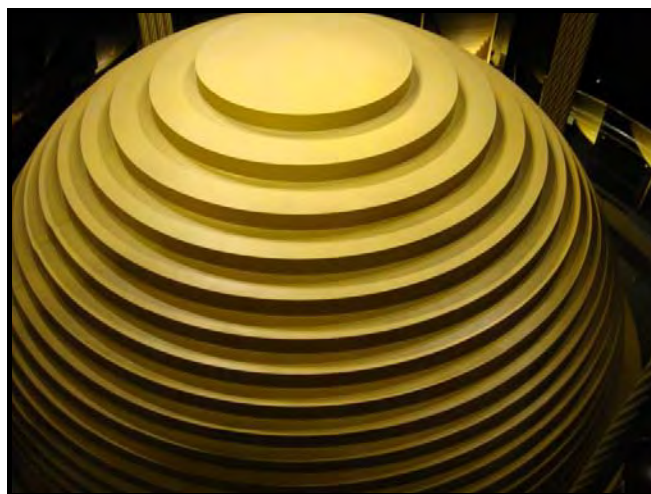


Figure 4-9 Tuned mass damper in Taipei 101, Taipei, Taiwan

City Hall Subway Apartment

As introduced in the previous section, Taipei is categorised as a city with high likelihood of severe earthquakes, as well as typhoons. This directly affects the structural design, especially tall building design. The critical loads for Taipei's medium to tall buildings are then typhoon and earthquake loads.

The building shown in Figure 4-10 is located near the City Hall subway station. It is a concrete frame structure with a hybrid bracing bend formed by V-bracing and the zipper columns.

According to R. L. Brockenbrough and Frederick S. Merritt (1999), V-bracing is classified as one of the concentrically braced frame. The bracing members of the concentrically braced frame act as a truss system to resist the lateral forces during earthquakes and heavy winds and are subjected primarily to axial forces in the elastic range. In severe earthquakes, significant inelastic deformation may occur in the bracing members, and this may lead the members into a post buckling stage due to the cyclic tension and compression. In this case, the concentrically braced frame is designed to avoid the preliminary failure of the structures.

V-bracing has the bracing connection at the mid-span of the beam. When under lateral loads, the two bracing elements act as compression and tension elements respectively. However, the tensile capacity of the bracing element is much higher than the compression capacity. Moreover, the unbalanced force at the beam intersection may cause beam yielding during severe seismic excitation. Consequently, the energy dissipation can be significantly increased whilst the damage to the floor system is also severe. In this case, if V-bracing is used to help resist lateral loads, strong beams having high flexural capacity to withstand the unbalanced forces are also required.



Figure 4-10 City Hall Subway Apartment, Taipei, Taiwan

However, if working together with the zipper columns, the disadvantages of the V-bracing system can be greatly eliminated. The zipper column is an alternative of the strong beams for the V-bracing system. When beams buckle, the zipper columns can transfer the unbalanced forces and distribute the inelastic deformation to other bracing levels so that the severe floor damage can be prevented.

In terms of the non-structural components, since this building was still under construction, the primary structure was completed at the time of visit, but only part of the glazing system was visible. It is assumed that pre-cast concrete panels might be most likely involved as infill walls. However, considering the feature of the primary structure, tolerances of non-structural components connected to the structure would be a concern for the designers and the builders.

Xinyi District Commercial Building

Figure 4-11 shows a commercial building which was still under construction at the time of the investigation. It is clear that the building is a composite frame structure with heavy bracing. Different to the building discussed in the above section, the structural frame of this building is composed of concrete columns and steel beams, and the bracing system in this building is zipper columns.



Figure 4-11 Xinyi District Commercial Building, Taipei, Taiwan

As discussed above, zipper columns can effectively distribute the beam deformation. However, they are normally used together with V-bracing or inverse V-bracing system. In this specific building, zipper columns are used on their own with the concrete frame and shear cores to resist the lateral movement of the building.

In terms of the non-structural components, since the building was still under construction at the time of the investigation, it was hard to judge the type and material of façades and infill walls.

4.2.3.3 Buildings in Beijing, P.R. China

The buildings investigated in Beijing are: China World Trade Center III, Jing Guang Center, and the Fortune Plaza Tower.

China World Trade Center III

The China World Trade Center III is a 330m-high, 80-storey building located in Beijing CBD. The architect of this building is Skidmore Owings & Merrill LLP. The structural and geotechnical design was carried out by ARUP Beijing.

From the discussion with the structural engineers in ARUP Beijing office, the main issue in the design of the China World Trade Center III was seismic resistance. Considering the geographical conditions of Beijing (discussed in previous sections), Beijing is partly in a high seismic region. Consequently, seismic design is one of the critical parts of the design process for buildings, especially tall buildings in Beijing.

The concept of the design of this building is that the width of the building decreases with the increase of the building height (Figure 4-12). It means that the column numbers need to be reduced upwards storey-by-storey. Under such circumstances, the seismic performance of the building needs to be carefully analysed during the design process. The designer finally chose composite steel walls as a core, composite columns and steel beams for the frame system, which work together with the bracing.

Regarding the non-structural components, the building has glass façades over its surface and it uses pre-cast concrete panels as partition walls. At the top levels, there are also truss-shaped concrete façades for decorative purpose.

According to the designer, even though the top façades of the building (truss-type façade) were originally considered as decoration, they were identified to have negative effects on the main structure under thermal loads. In the detailed design of the building, finite element modelling analyses were involved because of the complexity of the structure. The finite element models were developed and analysed under different loading conditions. The results revealed that when under thermal load, the movements of the top façades caused by the expansion and shrinkage of different parts can significantly affect the structural performance, especially the stress distribution in the adjacent components. Thus, the whole structure was re-analysed, by given serious considerations to the top façades, i.e. integrating them into the structural design of the building.



Figure 4-12 The China World Trade Center III, Beijing, P.R.China

Jing Guang Centre

Jing Guang Centre was built in 1990. It has 3 underground levels and 57 above ground levels. Its height is 208m. It had been the tallest building in Beijing for a long time.

Regarding the structure, Jing Guang Centre is a steel framed structure, with reinforced concrete shear walls. The bottom levels of this building use the steel reinforced concrete (SRC) to form the structural frame, making full use of the advantages of SRC structures such as high efficiency of concrete, low cost, outstanding seismic and fire resisting performance, and easy construction.

In terms of non-structural components, precast concrete panels are used as partition walls. The building also has elegant curving shape double glazed by the glass curtain walls from the base to the top (Figure 4-13).), which subsequently increases the cost and the difficulty of manufacturing and installation of the curtain walls, and thus in turn, increases the vulnerability of the façades under different loading conditions, especially when these glass curtain walls are connected to the main structure and work together with the primary structure as a system (as they are). However, based on the discussion with the structural engineers mentioned in previous sections, since the design of tall buildings in Beijing is dominated by high gust winds and the earthquake load, rigorous

design criteria on the serviceability of the building (typically stiffer structure) is adopted by the Chinese Standards. This to a large extent limits the chance of those glass panels to expose to large deflection introduced by the structural movement, and thus lowered the possibility of damages to these non-structural components, even these non-structural component are excluded to the design of the structure.



Figure 4-13 Jing Guang Centre, Beijing, P.R.China

Fortune Plaza Tower

As shown in Figure 4-14, the Fortune Plaza Tower in Beijing is a building with traditional square shape. It is 260m high, including 63 storeys. It is a typical reinforced concrete frame structure, with central cores and shear walls working together as its lateral resisting system. It has floor to ceiling windows all around the four sides, which means most of the outside walls are glass curtain walls. Pre-cast concrete panels are used as partition walls inside the building, whilst a great portion of areas are open areas because of the commercial purpose of the building. Similar to Jinguang Centre, the glass façades are the key non-structural components of this building and is vulnerable to different loads because of the large covering area and the very limited gap between each other.

This, again, together with the findings from Jinguang Centre, rise the question that should these non-structural components be integrated into the structural analysis to assess their vulnerability and/or to evaluate their structural contributions and the related cost savings.



Figure 4-14 Fortune Plaza Tower, Beijing, P.R.China

4.2.3.4 Buildings in Tianjin, P.R. China

As listed in Table 4-1, three buildings were investigated in Tianjin, the New Education Centre in Tianjin University, an anonymous residential building, and Tianjin Jiali Centre Office Building.

One of the major problems of the building design in Tianjin is its geotechnical conditions. Tianjin is famous for its soft clay foundation which is a big challenge to the structural engineers. Thus the geotechnical investigation and foundation design are always emphasized during the whole design process of a building.

The New Education Centre in Tianjin University, Tianjin

The new education centre in Tianjin University was built for teaching and learning purpose (Figure 4-15). Driven by its functions, large open spaces, large storey height, large door and window openings and efficient evacuation system are essential characteristics of the building. To achieve its functions, the building consists of a traditional concrete frame with shear cores as its primary structural system. The concrete frame can provide large open spaces as classrooms and multifunctional teaching spaces, and the service cores can be not only an integrated part of the lateral

resisting system, but also the most effective way of quickly evacuating people during rush hours.

Glass façade is used outside the building, similar to most of the modern tall buildings in China. The infill walls are built from pre-cast concrete panels and masonry.

Seismic design is also an important factor in the design of tall buildings in this city. However, unlike Taipei where the city has very high likelihood of severe earthquakes and typhoons, buildings in Tianjin normally do not adopt heavy bracing systems. Shear cores and strong frames are the commonly used lateral resisting systems.



Figure 4-15 The New Education Centre in Tianjin University, Tianjin, P.R.China

The non-structural components such as infill walls and façades are widely included in tall buildings in Tianjin. Pre-cast concrete panels and masonry walls are the norm for infill walls. Glass with aluminium frames composes the typical façade system for most of the tall buildings.

It is also noticeable that the way of building infill walls in China is different to that in Australia. In China, the masonry infill walls are built to fill the frame, with the very top layer bricks being inclined to one of the columns, on a 45 degree (approximately) gradient. By doing this, the pressure to the infill walls induced by the storey drift can be effectively eliminated. The details and diagram can refer to Chapter 7.

Jiali Centre Office Building, Tianjin

This building is a commercial building designed by ARUP Beijing office (2008). It is a 72 level building, with the height of 333m (Figure 4-16). The main lateral resisting system of the building is the braced steel frame with concrete shear core. The designed maximum storey drift is 450mm approximately.



Figure 4-16 Jiali Centre Office Building, Tianjin, P.R.China

After the discussion with the structural engineers, it is noted that the construction of infill walls in China is different to that in Australia and other places. In Australia, gaps between infill walls and the frame are specified in the structural design and are filled using elastic materials during construction. This, to some extent, reduces the chance of direct contact between infill walls and the structural frame, and thus provides a margin for the actual movement of the infill wall. In China, the masonry infill walls are built to fill the frame, with the very top layer of bricks being oriented along an in-plane 45 degree diagonal line (approximately). By doing this, the forces/stresses transferred from the frame to the infill walls (if there is any) introduced by the storey drift can be effectively dispelled.

Tanggu Apartment, Tanggu District, Tianjin

Shown in Figure 4-17, the Tanggu Apartment building is a typical residential tall building in China, adopting square-shaped reinforced concrete frame with cores as its primary structure. Pre-cast concrete panels and masonry are used for infill walls, whilst claddings can be found on the surface of the outside walls. The floor plan of this type of building is normally regular throughout the building, designed for residential purposes.



Figure 4-17 Tanggu Apartment, Tanggu District, Tianjin, P.R.China

Considering the location and the geological conditions of the building, because it is located near the harbour, strong wind is expected. Also, because of the special soft clay ground condition and its high seismic hazard feature, the design of the building focused on the stability and the strength of the structure.

Moreover, the foundation design of tall buildings in Tianjin is always a bottle neck to most of the structural designs. Deep pile foundation or pile foundation with underground aligning walls and plates are widely adopted in the current construction industry in Tianjin. These types of foundations can efficiently solve the problems such as the uneven settlement caused by the soft clay ground condition and the seepage of underground aligning walls caused by the high underground water table.

4.2.3.5 Buildings in Dalian, P.R. China

Dalian is a coastal city lying in the northern part of China. As discussed in the previous section, it is neither within high seismic hazard region nor in the high wind region. Hence the design emphasis differs from that in Beijing and Tianjin. The buildings investigated in this city include: The Hope Mansion, Ganjingzi District Apartment and Xinghai Guobao Residential Buildings.

The Hope Mansion

The Hope Mansion building is a 170m-high building, with 41 floors including 3 basement levels (Figure 4-18). It is a reinforced concrete structure, including a great portion of prestressed components. The lateral resisting system of this building is concrete core and frame with main supporting columns at the four corners of the building and a narrow base at the bottom as shown in Figure 4-18.

Checking the geographical conditions of Dalian, it is noted that this is not a city with severe seismic hazard or in a windy area. Hence, the requirements of the rigidity and ductility of the structure are not as high as that of the buildings in other cities such as Taipei and Beijing. The seismic resisting level of this building was designed as level 8 according to the Chinese design standards.

Non-structural components involved in this building were precast concrete panels as partition walls and glass panels vertically meshed by the surface concrete frame as façade.



Figure 4-18 The Hope Mansion, Dalian, P.R.China

Ganjingzi District Apartment Building

The apartment building located in Ganjingzi District in Dalian was still under construction at the time of the investigation. From the picture shown in Figure 4-19, it is clear that the building has a concrete frame structure with shear walls as its main lateral resisting system, similar to the Tanggu Apartment in Tianjin. The structural forms of these two buildings are typical of most residential buildings in China.



Figure 4-19 Ganjingzi District Apartment Building, Dalian, P.R.China

Xinghai Guobao (National Treasure) Residential Buildings

The Xinghai Guobao residential buildings are located on Xinghai Square, Dalian. Xinghai Square sits in the north of Xinghai bay, with the shape like a giant-star. The Xinghai Guobao residential buildings are located at the edge of the square. As shown in Figure 4-20, both the modern characteristic and the traditional Chinese cultural element are combined in the architectural design of these buildings, making them elegant and outstanding.

From the structural perspective, these buildings are designed especially as top class accommodation. Hence, being spacious, comfortable, safe, and secure are important factors apart from its impressive presentation. This results in the inclusion of the concrete frame, shear walls, and concrete cores in their primary structure systems. For the secondary structural elements of these buildings, claddings, infill walls, floor-to-ceiling windows are all widely adopted.



Figure 4-20 Xinghai Guobao Residential Building, Dalian, P.R.China

4.2.4 Discussion and Comparison

The above sections introduce the buildings investigated in cities from Australia, Taiwan, and mainland China. From the discussion, it can be identified that buildings in different areas have their own design features, which are determined by complicated factors, such as local culture, climate, geographical condition, political reasons.

In Australia, due to the low probability of seismic hazard in most of the areas, especially in the three cities investigated, wind force governs the lateral design of high-rise buildings in most of the cases. Concrete and steel frame structures are commonly identified in Australia, having large amount of glass façade and partition walls adopted in both commercial and residential buildings. According to the discussion with structural engineers in Australia, even though the individual non-structural components are designed in detail according to specific standards, they are considered isolated from the primary structures and thus are not integrated in the structural design analysis.

Dominated by the seismic and wind design, the design of tall buildings in Taiwan and mainland China mainly focus on the primary structures. No matter what type of structural form is adopted, the core strategy of their design is to assure the stability and ductility of the primary structure, i.e. the structural frame, the core, the whole lateral

resisting system, to make sure the primary structure of a building will not be damaged during disasters such as severe earthquakes and the high wind. In terms of the secondary elements, although infill walls and façade are widely used in the tall buildings in these cities, they are seldom included in the whole design analysis. From the investigation, there are several reasons of the exclusion of the secondary elements in the structural design in Taiwan and mainland China.

- During the severe earthquakes and high wind attack, damage of those non-structural components is inevitable. To be more cost-effective, both the designer and the client would not spend more time and money on developing/integrating the secondary elements into the structural analysis, even though it might be beneficial from the long term point of view;
- Extra rigidity and ductility have been designed for the primary structure for the worst load cases. Thus, when under service load, the overall movement of the structure will be much less than that of the building designed in the non-hazard areas. That directly leads to the diminishing of the interactions between the primary and secondary structural elements whereupon eliminating of the damage/influence on the secondary elements;
- Different approaches are used in the construction of the secondary elements. For example, as discussed in Section 4.2.3, in China, the masonry infill wall is built with the top layer brick lying on a 45 degree gradient. This can effectively eliminate the pressure transferred from the frame deformation, which to some extent isolates the non-structural components from the primary structure.

All in all, for the 15 buildings in 7 cities, even though it is hard to do a detailed and thorough analysis and the comparison, some points are summarised in the following sections.

4.2.4.1 Buildings in Australia

According to the discussions on the three buildings investigated in Australia, as well as the wide observations and communications with local industry, the following features can be summarised for the high-rise buildings in Australia:

- Steel and reinforced concrete structures are both widely adopted in the design of high-rise buildings;

- In the design of high-rise buildings, wind load normally governs in terms of the stability and serviceability of the structure due to the low seismic hazard level in most part of Australia;
- Concrete core, shear walls, and structural frame are most commonly adopted lateral resisting systems;
- Glass façades are involved in most of the commercial buildings;
- Precast concrete panels and masonry walls are used as partition walls/infill walls for buildings and they are considered as non-load bearing components which are isolated from the primary structure;
- The façade system is designed separately by façade engineers and is considered detached from the structure in the structural design.

4.2.4.2 Buildings in Taipei, Taiwan

From the investigation of the 3 buildings in Taipei, it can be concluded that the tall buildings in Taipei have the following common characteristics.

- Regardless of the different structural forms, all the tall buildings are designed mainly to resist the seismic and typhoon loads;
- Braced frames are the widely used structural form for tall buildings in Taipei. Even though different types are chosen according to the specific requirements of different buildings/structural presentations, bracing is very popular in tall building design in Taipei because of its capacity to provide extra ductility and extra stability to the structural frame;
- Both concrete and high-performance steel structures are common in Taipei;
- Modern glass façade/curtain walls, pre-cast concrete infill panels are the commonly used secondary elements for decoration and thermal purposes.

4.2.4.3 Buildings in Mainland China

Compared with other cities, the three cities in China have many more super tall buildings. However, in terms of the structural features, a lot of similarities can be found.

The structural features of tall buildings in mainland China can be summarised as follows:

- The dominant lateral loads in tall building design in China are earthquake load and wind load;
- Frame structures with concrete cores are the most common structural form used in tall buildings in China whilst bracing systems can also be easily identified from many buildings;
- Precast concrete panels are normally used as wall panels, and masonry infill walls are also widely used in tall buildings;
- Claddings in various materials such as glass and ceramic, are common elements of tall buildings;
- Many commercial buildings in China have modern façade systems appearing outside of the buildings. Glass panels with aluminium frames are the common types of façades;
- The façade systems in China are mainly considered for the thermal purposes. In other words, the main load-bearing consideration of the façade design is thermal loads, if there is any;
- The diversity of foundation conditions makes the design of tall buildings even more challenging in China.

4.3 Summary of Findings and Gap Analysis

Based on discussions in previous sections, the structural design of high-rise buildings is greatly influenced by the environment, local geological conditions and culture.

It can be concluded that even though the structural forms of tall buildings are more or less similar in different regions, subtle variations of the design features exist because of the complexity of the geological conditions and the hazard levels.

Many non-structural components are involved in the buildings in each city. Glass façade, precast concrete partition walls and masonry infill walls take great portion of the

components of a building. Moreover, these non-structural components are all well designed as individual components separate to the structural design according to various standards.

According to the investigation, although most of the non-structural components are well connected to the main structure according to the requirements of different standards (e.g. AS/NZS 1170.0:2002) as they appear at the service stage of the building, few of them are considered in the holistic structural analysis during the structural design.

It is clear that current design analyses of tall buildings in different regions do not show enough correlation with the construction practice in terms of interactions between the primary structure and non-structural components.

4.4 Scope and Limitations of the Study

As discussed in Chapter 2, there have been sophisticated analyses and design philosophies for structural elements under different loading conditions existing for a long period of time. Thus, this study focused on the integration of non-structural components into the structural analysis, the differences of the structural performance caused by including those non-structural components in the analysis, and the subsequent effects on the non-structural components.

The investigation of buildings in this chapter shows that different regions have different design focus based on the specific consideration of the local geological condition, environment, and perhaps cultural issues.

In Australia, since there is very low likelihood of earthquakes and cyclones in most part of the country, the design of a high-rise building is mainly governed by wind load. Moreover, to a great extent, the serviceability (or say, the lateral drift) of the building dominates the behaviour and the design of tall buildings. Thus, to step pass the gaps identified in previous section, the evaluation of the influence of integrating non-structural components into the structural analysis on the overall structural performance is paramount to local design practice.

However, in Taiwan and mainland China, due to the high seismic hazard level and some other extreme loading conditions, such as the frequent attacks by typhoon, the design of tall buildings is governed by these extreme loading conditions and the strength and

stability of the structure are the dominant factors in building design. Hence, the potential contribution of integrating non-structural components into the structural analysis to the overall building performance is less relevant as the stiffness of the structural skeleton dominates the performance of the building in these regions. On the contrary, damage to the non-structural components caused by the interactions between the primary structure and non-structural components during the service life of the building are worth investigating.

The scope and limitations of this study, based on the above discussion, can be concluded as:

- The analysis carried out in this study focuses on the practice in Australia or areas with similar geological conditions and hazard levels;
- It is mainly limited to the serviceability design of tall buildings;
- Key non-structural components such as façade and partition walls will be analysed and used to identify the influence of non-structural components on the structural performance.

On the basis of the above scope and limitations, detailed analysis of a case-study building, laboratory tests and the parametric study are conducted (Chapter 5, 6, 7) to achieve the aim and objectives of this research.

CHAPTER 5

PRELIMINARY FINITE ELEMENT ANALYSIS OF A CASE-STUDY BUILDING

5.1 Introduction

Chapter 4 provides selected samples of typical tall building designs in particular countries. The observations led to the conclusion that the “as constructed” buildings had non-structural components durably fixed to the skeletal structure, which is contrary to design analysis.

This chapter investigates the building system of their connectivity through the analyses of a case-study building, Dock 5, one of the 15 buildings of the field reconnaissance. As introduced in Chapter 4, this building is located in Dockland, Melbourne, Australia. It is a 32-storey reinforced concrete structure, with core and shear walls integrated by flat plates as its lateral resisting system.

This chapter provides the detailed discussion of the case-study building analysis. To analyse the building, main design drawings were reviewed, facilitated by visiting the construction site and communicating with design engineers. On the basis of the information obtained from the communication and observations, the design features of this building, the details of different non-structural components and diverse connections between both structural and non-structural components become the focus of the analyses.

Finite element models were developed based on this case-study building. Preliminary analyses were carried out aiming to assess the difference caused by integrating non-structural components into structural analysis to the overall structural performance. The key non-structural components adopted based on the case-study building are precast infill walls and partition walls, and outrigger glass façades. In the analyses, the influence of different non-structural components was evaluated. Other factors, such as quantity of non-structural components and connections between primary structure and non-structural components were also investigated.

5.2 Investigation of the Case-study Building

The case-study building is the Dock 5 building, a 32-storey high-rise residential building located in dockland, Melbourne, Australia. As introduced in Chapter 4, this building was developed by Bovis Lend Lease Pty. Ltd. The architects of this building are John Wardle Architects and HASSELL - Architects in Association, and the structural consultants are Arup and Connel Wagner. It is the winner of 2008 RAIA Best Overend Award for Residential Architecture - Multiple Housing (VIC).

The 32-storey main building is a reinforced concrete apartment building with a concrete core and shear walls integrated by flat slabs as its lateral resisting system. Figure 5-1 shows the typical floor plan of this building. Apart from the main building, there is another 9-storey reinforced concrete apartment building adjacent to it. In this study, only the 32-storey main building was analysed.

5.2.1 Aim and Objectives of the Analyses

In Chapter 2, detailed review of different types of connections was provided. It is widely agreed that regardless of the diverse presentations, there are three typical types of connections based on their rigidity: rigid connections, semi-rigid connections, and pinned connections.

The role played by structure connections is critical to the structure behaviour. Connections between structural elements such as beam-column connections have already been widely developed and analysed. However, even there is some evidence illustrating that non-structural components will dramatically influence the lateral performance of high-rise buildings, few investigations have been conducted to connections between non-structural components and the primary structure. This chapter is therefore focusing on this special interface. The following three main objectives were to be achieved at the completion of the case-study building analysis:

- To identify different types of structure intersections of Dock 5;
- To clarify the properties of different types of connections;
- To quantify the influence of connections to the lateral performance of the structure.

5.2.2 Structural Form and Construction Materials

Figure 5-2 shows the typical core plan of the Dock 5 building. It is identified that a reinforced concrete service core and two sets of shear walls compose the building lateral resisting system. Moreover, integrated by the pretension concrete slabs, this structure performs as a box system to resist lateral load.

Main materials used in this building are reinforced concrete and steel. Precast concrete panels and masonry walls are designed as partition walls. Glass panels and metal frame are used for the façade system. Depending on different types of elements, the strength of the concrete used in this building varies from $32MPa$ to $80MPa$, and the diameter of reinforcement steel bars is from N12 to N36.

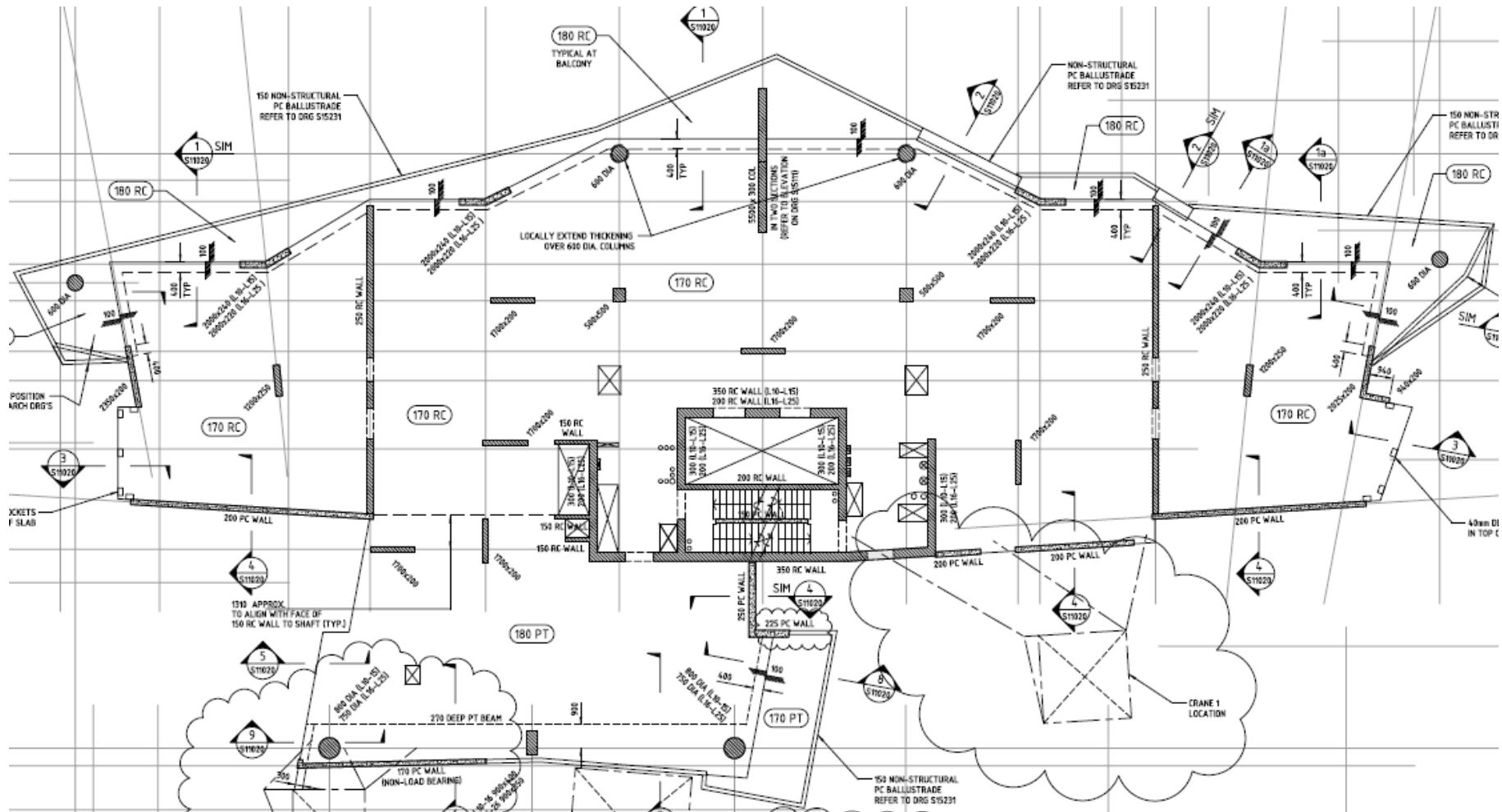


Figure 5-1 Typical floor plan of Dock 5 32-storey main building

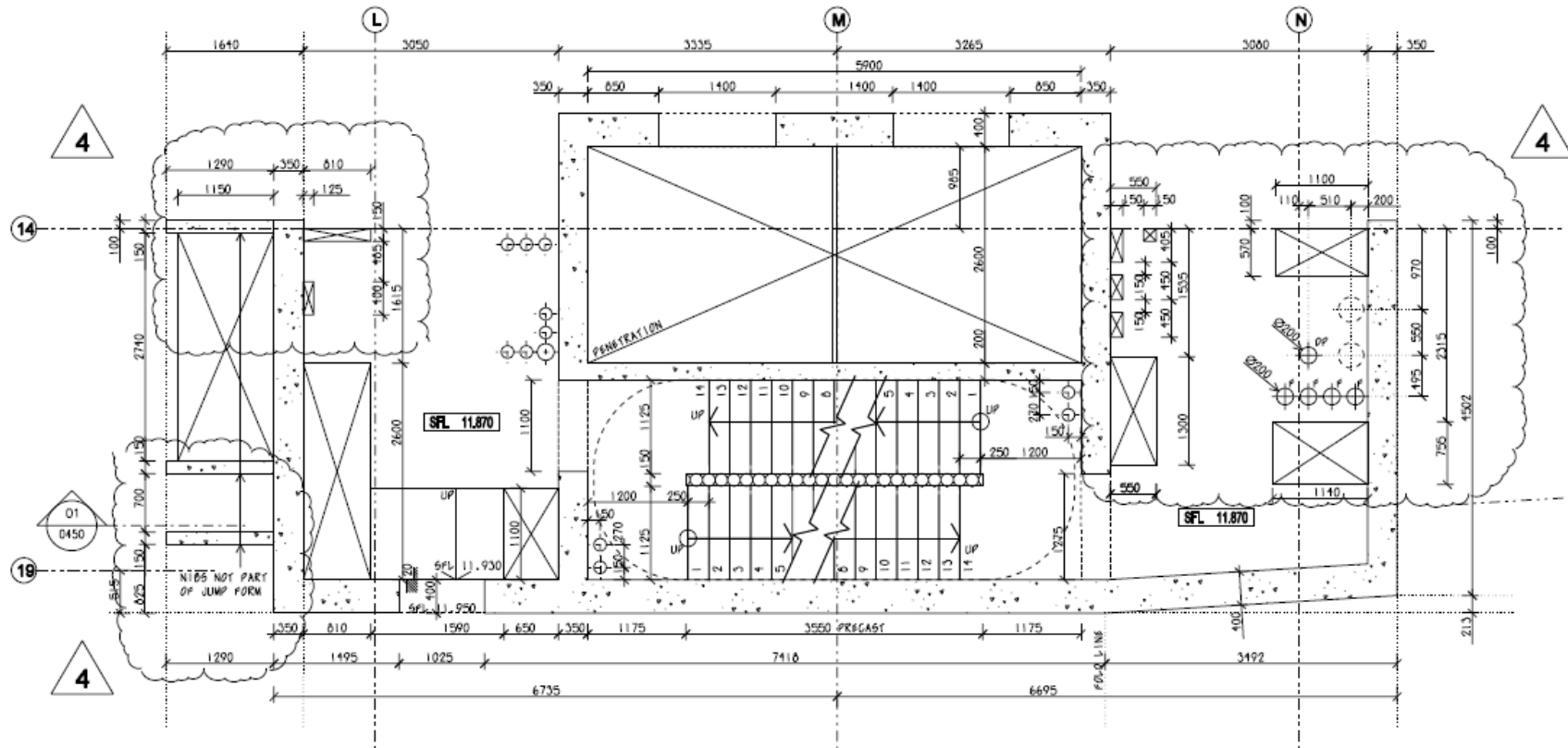


Figure 5-2 Core plan of Dock 5

5.2.3 Details of Different Components

5.2.3.1 Structural Components

The main structural components mentioned in this study are the components composing primary structure. It includes beams, columns, floor slabs, and shear walls. The design details of these components are in accordance with the requirements of AS 3600 (2001). Some of the details of the structural components are shown in Table 5-1.

Floor slabs

Floor slabs are the main horizontal components that transmit both the live loads and the dead loads to the vertical framing supports of a structure (Nawy 2003). In this building, the thickness of a typical floor slab is 190mm. Most of the slabs are reinforced concrete slabs, with the concrete strength of 32MPa. Both top and bottom reinforcement are designed for the slabs. The diameters of reinforcement steel bars vary from N12 to N36. Some pretension slabs are also used with different the concrete strength of 40MPa.

Beams

Beams are the structural components that transmit the tributary loads from floor slabs to vertical supporting columns (Nawy 2003). There are two types of beams designed in Dock 5 building: normal beams and band beams. The band beams are cast monolithically with the slabs working as T-beam or L-beam. Similar to floor slabs, steel reinforcement are designed to all the beams. The strength of the concrete is 32MPa and the diameter of reinforcement steel bars varies from N16 to N36.

Columns

Columns are the vertical elements which support the structural floor system and transmit axial compressive loads, with or without moments (MacGregor 1997; Nawy 2003). In this building, most of the columns have rectangular cross sections. All of the columns are reinforced concrete columns with concrete strength from 32MPa to 80MPa and the diameter of reinforcement steel bars from N16 to N36.

Shear walls

Shear walls are concrete structural concrete walls to resist lateral loads. The thickness of reinforced concrete shear walls in the Dock 5 building varies from 350mm to 400mm.

Strength grade of the concrete used for shear walls is $80MPa$. The diameter of the reinforcement steel bars is from N16 to N36.

Table 5-1 Details of structural elements

Element	Cross section mm^2	Thickness mm	Re-bar Diameter mm	Concrete grade MPa
Beam	T- L-		N16~N36	32
Column	Rectangular		N16~N36	80~32
Floor slab		190	N12~N36	32~40
Shear wall		150~400	N16~N36	

5.2.3.2 Non-structural Components

The key non-structural components identified in this case-study building are infill walls, partition walls, façade panels, doors, windows, and stairs. They are designed as non-load bearing elements. The details of these components are depicted as follows.

Infill walls and partition walls

Infill walls and partition walls in the Dock 5 building are brick or precast concrete infill walls and precast concrete partition walls. These walls are thinner than the reinforced concrete structural walls. The thickness of the infill walls and partition walls are from $100mm$ to $200mm$. The concrete strength is $40MPa$, with reinforcement if required. All the details of the precast panels are in accordance with AS 3600:2001 (2001), AS 3850:2003 (2003), AS/NZS 1554:2008 (2008).

Façade

Apart from the aesthetical function, the non-structural glass façade panels play an important role in protecting the building from the outside noise and the thermal load. The Dock 5 building adopts double-skin modern façades, using coloured glass panels meshed by metal frames (mainly 150UC37).

Doors and windows

Doors and their frames in this building were designed as timber doors with timber frames. Windows are formed by single or double-skin glasses together with metal

frames. Because of the residential purpose of this building, most doors and windows in this building have uniformed dimensions and properties.

5.2.4 Connections between Components

Generally, connections in the building can be categorised according to different types of components (Table 5-2 to Table 5-4):

- Connections between structural components (Table 5-2). These are the connections between beams, columns, walls and floor slabs;
- Connections between structural and non-structural components (Table 5-3). This type of connection is the interface between structural and non-structural components, through which the non-structural components are connected to the primary structure;
- Connections between non-structural components (Table 5-4). These are connections between different non-structural components. For instance, the connections between façade panels and the façade frame.

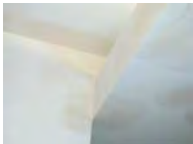

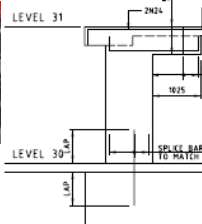

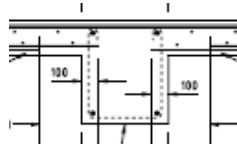
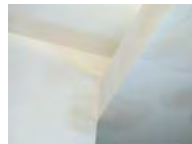
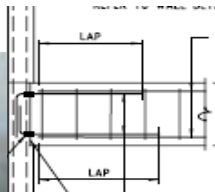

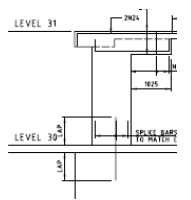

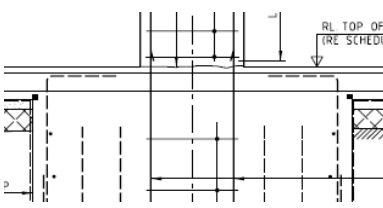

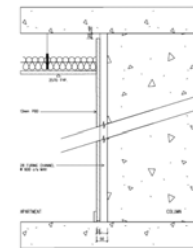
As identified in the previous section, the purpose of this analysis is to investigate the influence of different connections on the overall performance of high-rise buildings under serviceability loads. Under this condition, the probability of the appearance of the plastic hinge at reinforced connection points and the cracking problem of concrete in tension can be ignored. Consequently, connections between reinforced concrete structural elements can be considered as rigid connections.

Since structural components in this building are mainly reinforced concrete elements whilst non-structural components such as partition walls, façades, windows and doors are precast concrete panels and glass or timber panels with metal frames, the connections between structural and non-structural components of the Dock 5 building are mainly bolted connections. This type of connection can directly influence the load transfer between structural and non-structural components, and thus, the connection properties become an important index to the contribution of non-structural components to the structural performance. Different connection properties may lead to different loading and failure mechanisms. Under this circumstance, it is critical that the connections between structural and non-structural components are analysed in detail.

The connections between different non-structural components are metal-to-metal, metal-to-timber, or metal-to-glass connections. Metals are always connected together by bolts or welding. Bolts, screws and nails are commonly used to connect timber components. Even though such connections may influence the stiffness contribution of non-structural components, regulated by the detailed operation and installation standards from the manufacturers, they usually become the influencing factor of the stiffness of each single non-structural component as one of the properties listed on the specification sheet provided by the manufacturer.

Hence, the focus of the detailed analysis in this study is the connections between structural and non-structural components.

Table 5-2 Connections between structural elements

	Beam	Column	Slab	Wall
Beam	<ul style="list-style-type: none"> • Double reinforced and continuous • Beam to beam connections are made through crossing rebar and concrete coating 	<ul style="list-style-type: none"> • Rigid connection • Beam to column connections are made by high tensile bars  	<ul style="list-style-type: none"> • Rigid connection  	<ul style="list-style-type: none"> • Semi-rigid connection  
Column	<ul style="list-style-type: none"> • Rigid connection • Beam to column connections are made by high tensile bars  	<ul style="list-style-type: none"> • Columns are always under compression and are classified as short or long and slender columns • Column to column connection made by dowel bars 	<ul style="list-style-type: none"> • Rigid connection  	<ul style="list-style-type: none"> • Rigid connection  

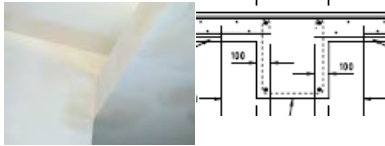
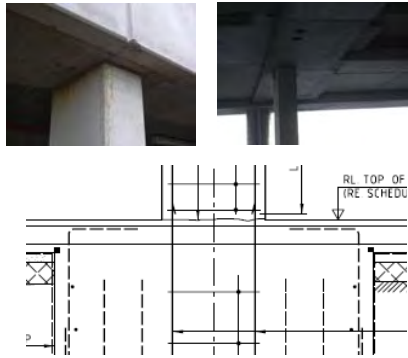
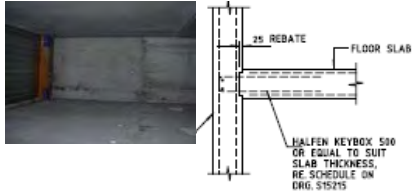
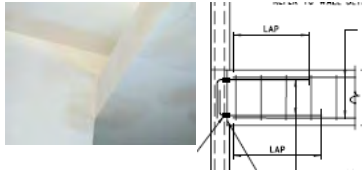

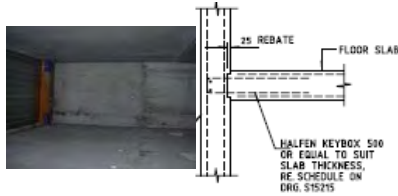
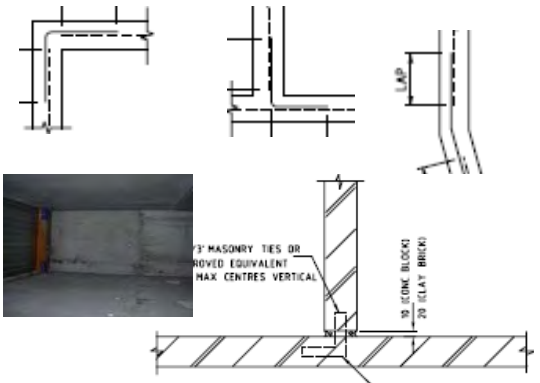
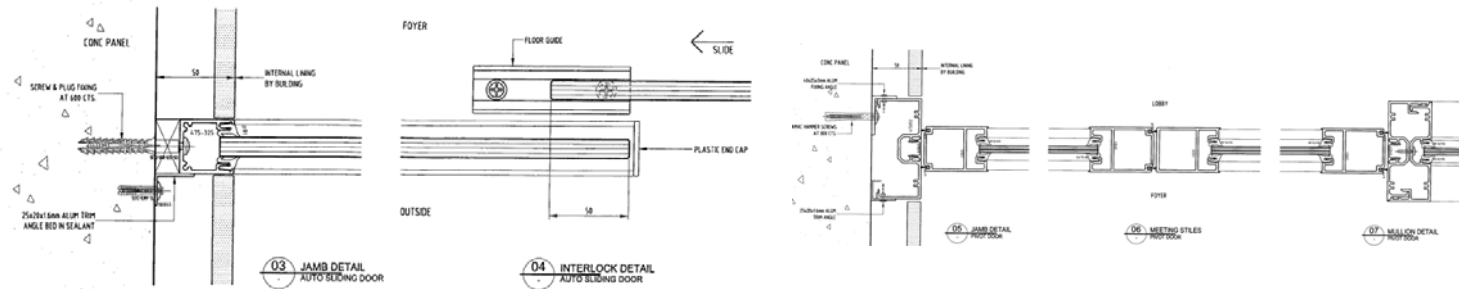
	Beam	Column	Slab	Wall
	<ul style="list-style-type: none"> • Rigid connection 	<ul style="list-style-type: none"> • Rigid connection 	<ul style="list-style-type: none"> • One-way, two-way • Double Reinforced 	<ul style="list-style-type: none"> • Semi-rigid connection
Slab				
	<ul style="list-style-type: none"> • Semi-rigid connection 	<ul style="list-style-type: none"> • Rigid connection 	<ul style="list-style-type: none"> • Semi-rigid connection 	<ul style="list-style-type: none"> • Rigid, semi-rigid, and pin connection
Wall				

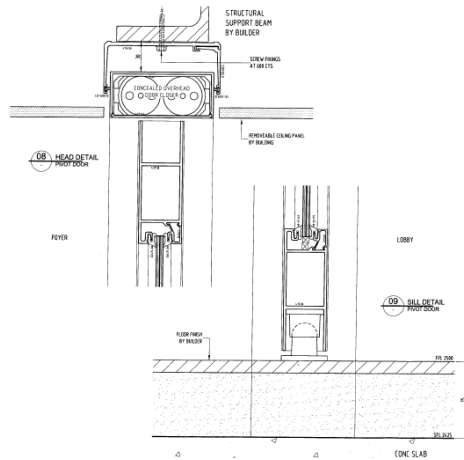
Table 5-3 Connections between structural and non-structural elements

Structural Components

Connection Type A



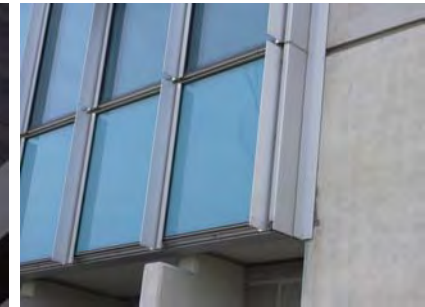
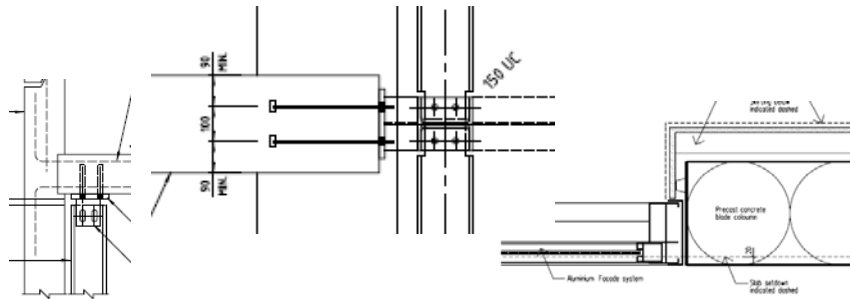
Windows and Doors



Structural Components

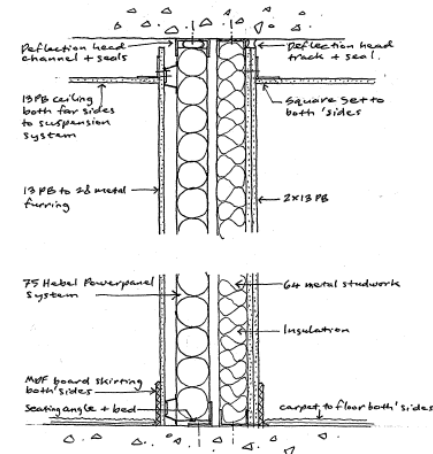
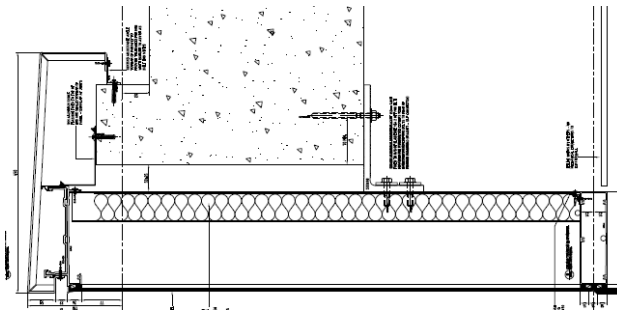
Connection Type B

Façade



Connection Type C

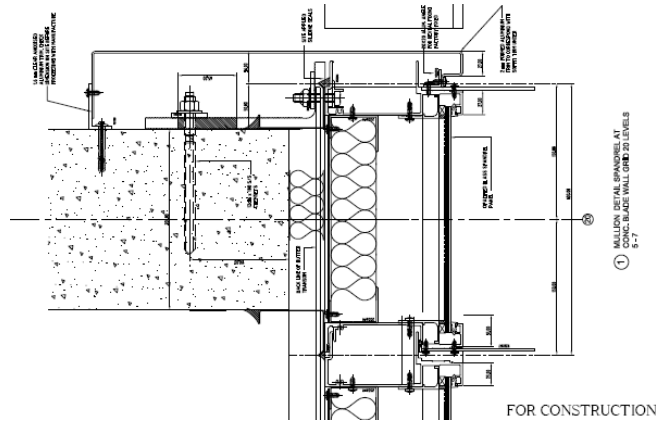
**Partitions
Infill wall**



Structural Components

Connection Type D

Curtain wall



Connection Type E

Stairs

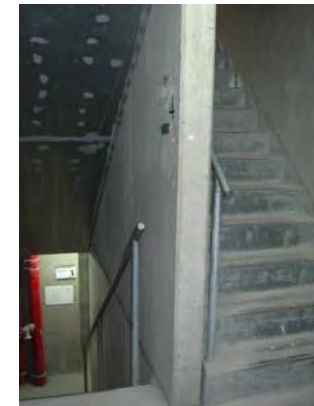
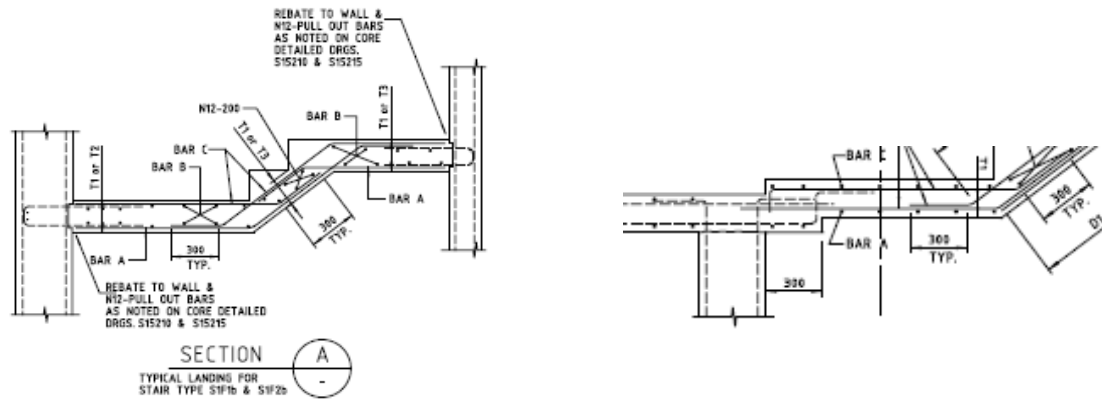
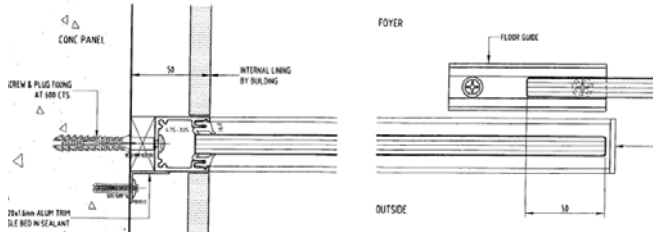
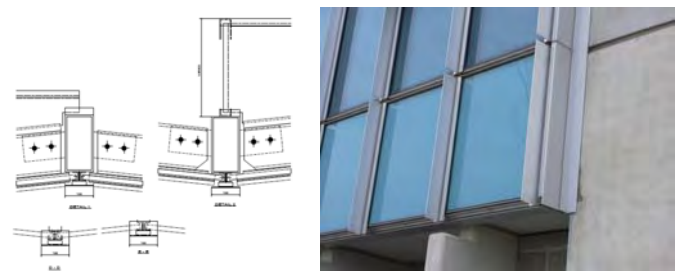
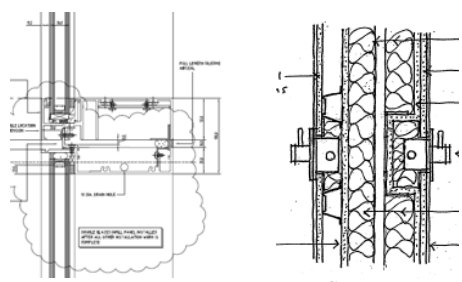
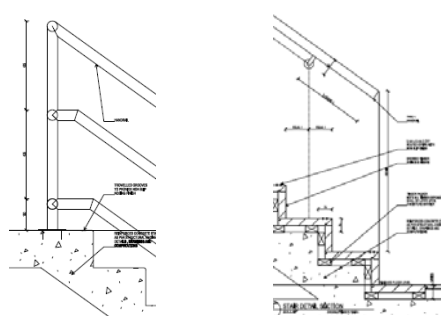


Table 5-4 Connections between non-structural components

Inter connection	
Connection Type I	
Windows	
Connection Type II	
Façade	
Connection Type III	
Partitions Infill walls	
Connection Type V-IV	
Stairs	

5.3 Preliminary Finite Element Analysis

As discussed in previous sections, connections between structural and non-structural components were required to be analysed. Based on the above tables and design details of the building, three types of connections were further investigated.

- Connections between different materials. For instance, connections between the metal frame of façade system and the reinforced concrete primary structure. A typical example of this type of connection is shown in Figure 5-3. The non-structural components are welded to the steel plies which are bolted to concrete structural components. Hence, with the change of the bolting arrangement, the rigidity of the connection can vary from rigid to semi-rigid, or even to pinned;

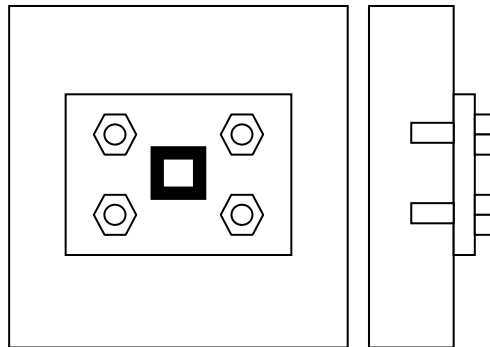


Figure 5-3 The composite connection

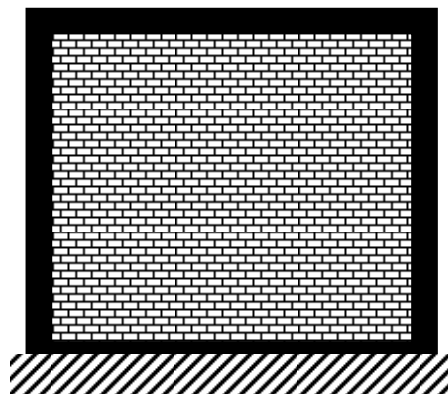


Figure 5-4 The non-structural wall

- Connections between metal materials. Some of the non-structural components are connected to steel beams either by bolts or by the welding. The bolting or welding details are critical to the rigidity of connections. Similar to connections between

different materials, there are also three potential properties of this type of connection: rigid, semi-rigid, and pinned;

- Connections between partition walls/infill walls and the structural frame. Partition walls and infill walls are typical non-structural components. In some cases, these non-structural walls are built in the structural frame as built-in walls whilst in other cases, small gaps between non-structural walls and the frame exist. The simplified expressions of the connections between the structural frame and the non-structural walls are shown in Figure 5-4.

Finite element models were developed to analyse the structural performance with and without the typical non-structural components in this case study building. Moreover, the influence of the quantity of the non-structural components and the properties of connections between structural and non-structural components were also investigated. As discussed in Chapter 2, many studies conducted finite element analysis on the behaviour of structures (Mahendran and Moor 1999; Grierson and Khajehpour 2002; Kicinger et al. 2005; Sarma and Adeli 2005; Wu et al. 2006). It is concluded that 3-D models can represent the real behaviour of the building more precisely than 2-D models. Thus, in this study, 3-D finite element models were developed.

5.3.1 Software

The commercial finite element analysis software ANSYS 10.0 (University Introduction Version) was used as an analytical tool in this study. There are several main advantages to choose this software package:

- The powerful element library assures different elements in a building system can be defined precisely and sufficiently;
- The comprehensive material library provides both standard material properties and the self-defined details for different materials;
- The capacity of the software in defining the degrees of freedom for each single node provides possibility to investigate the effects caused by changing of connection details;

- The sophisticated analysing system designed in the software makes it possible to analyse the model in different scenarios (e.g. the linear and non-linear analyses, small and large deflection effect, static and dynamic analyses.).

On account of the merits briefly described above, ANSYS is capable of defining and solving the problems rose in this study. Hence, 3-D finite element models were developed by using ANSYS to facilitate the analyses of the building performance with and without different non-structural components and the influence of connection properties.

5.3.2 Understanding the Failure Mechanism of Connections

Even though the chance of the failure of connections between structural and non-structural components is rare, typical failure modes of different types of connections are still discussed to better understand the interactions between structural and non-structural components.

Based on the review of current research, failure mechanisms of composite connection, mental connection and connections between structural frame and infill walls are summarised in the following sections.

5.3.2.1 Composite Connections

In the composite connection, bolts are used to fasten the steel ply and concrete structure. According to Kabche et al. (2007), there are literally three types of failure in composite connections:

- Concrete failure. It happens due to the failure of concrete bearing capacity. Under this scenario, the connection fails by the crushing of surrounding concrete as shown in Figure 5-5;
- Failure of the steel ply. It is mainly caused by the stress concentration near the hole area. Both insufficient material strength and flaws can lead to this type of failure. The failure mode is shown in Figure 5-6;
- Failure of the bolt. It normally happens because of the inadequate bearing capacity of the bolt. Similar to the failure of the ply, if the bolt can not withstand the design

load, or is not manufactured up to the required standards, when subject to the excessive loading, the bolt fails (Figure 5-7).

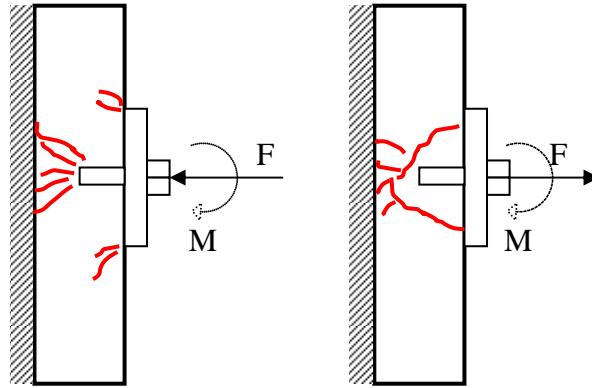


Figure 5-5 Concrete failure in composite connection

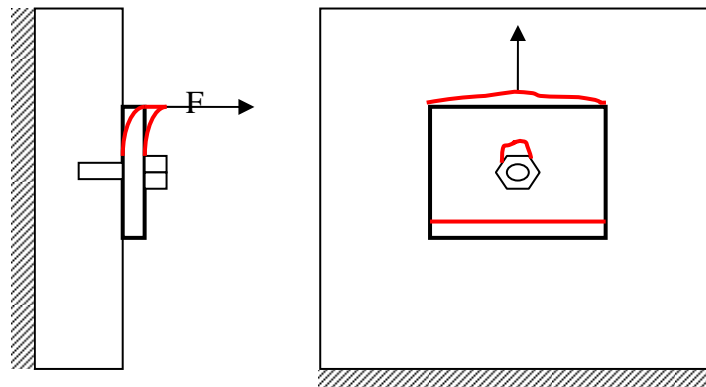


Figure 5-6 Steel ply failure in composite connection

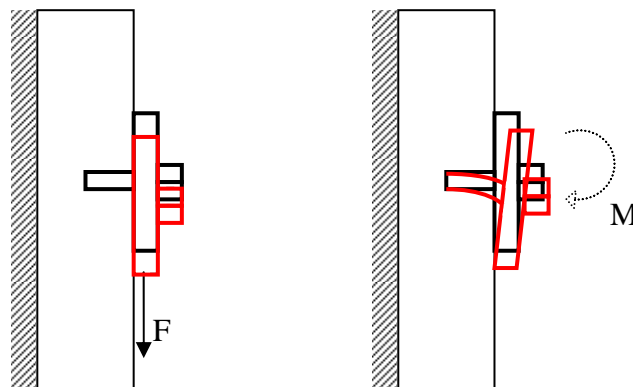


Figure 5-7 Bolt failure in composite connection

5.3.2.2 Connections between Metal Components (Steel Connection)

In most cases, metal components (mainly steel components) are connected by bolts. According to Casafont et al (2006), Rodrigues et al (1998) and Olsen (2001), failure mechanisms of this type of connections include:

- Failure of the ply. Similar to composite connections, it is mainly tearing failure (Figure 5-8);

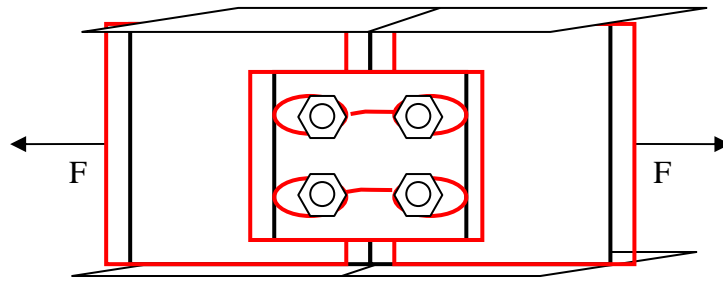


Figure 5-8 Ply failure in steel connection

- Failure of the bolt. Due to the insufficient strength or the extra stress caused by the imprecise installation, bolts will fail when the shear force or bending moment exceeds their bearing capacity (Figure 5-9);

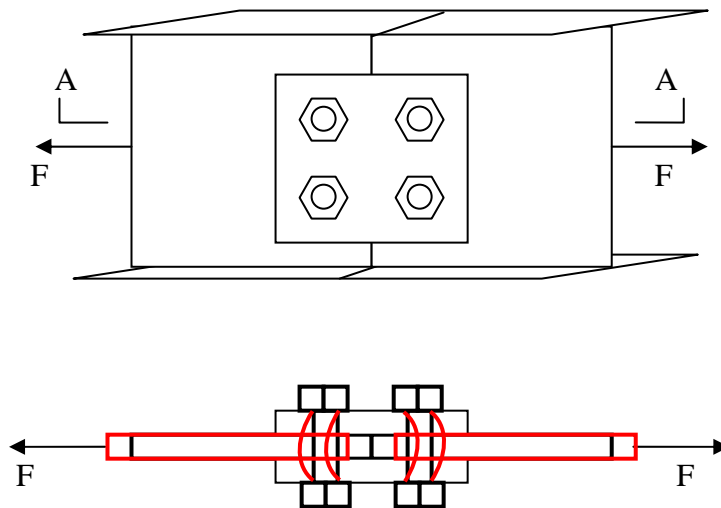


Figure 5-9 Bolt failure in steel connection

- Failure of components. The component fails when it cannot withstand the excessive loads because of the improper design or the defections of materials. Usually this type of failure occurs under ultimate loads.

5.3.2.3 Connections between Structural Frame and Infill Walls/ Partition Walls

In the construction practice, infill walls or partition walls are normally connected to the structural frame by using elastic fillings to fill the small gaps between the wall and the frame. Sometimes, they are also treated as built-in walls. Different from failures of other types of connections, the failure of the connection between infill wall and the structural frame leads to the contact between the two parts and thus the change of the load path (as

shown in Figure 5-10). The two extreme scenarios of this type of connection are no connection (gaps exist between the structural frame and the infill/partition wall) and rigid connection (the infill/partition walls are constructed as built-in walls).

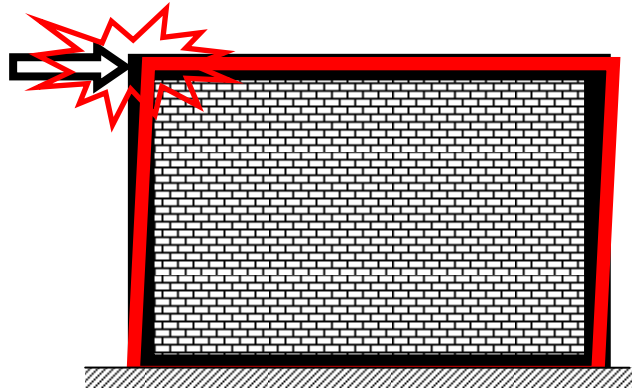


Figure 5-10 Infill wall

5.3.3 Modelling Details

Based on the understanding of different failure modes of connections discussed in previous section, the scope of this analysis was refined. Detailed discussion on the modelling procedure and assumptions on materials and analysing approach are provided.

5.3.3.1 Properties of Structural and Non-structural Components

As discussed in Section 5.2.3, diverse structural and non-structural components are involved in this building. The primary structure of this case-study building composes of a concrete core and two sets of shear walls. These two components are integrated by the flat-plate floor system working as a box to resist the lateral loads. In the modelling process, the design details of the case-study building were adopted. A 12m by 6m concrete core and two sets of shear walls which are 12m away from the core on each side were modelled, as shown in Figure 5-11. Thicknesses of the core wall and shear wall are 0.4m and 0.3m respectively. The floor slab is 0.2m thick. In the finite element model, a 3-D 4-node shell element, SHELL63, was selected to represent the walls and the floor slabs.

In terms of non-structural components, the precast concrete partition wall has a thickness of 0.2m. The façade system includes the 0.01m thick glass panels and the metal frame made from universal columns 150UC. Shell elements (SHELL63) were also engaged in the modelling of glass panels whilst a 3-D 3-node beam element, BEAM4, was employed to model the façade frame.

Details of the components are listed in Table 5-5.

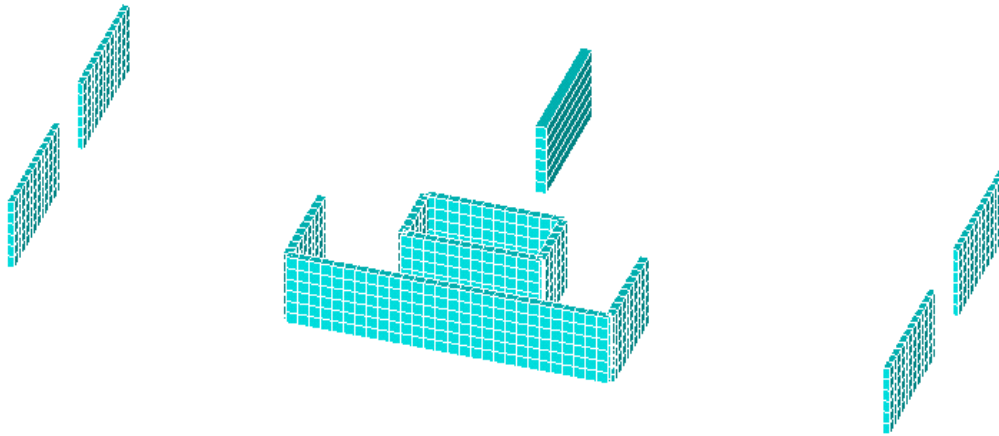


Figure 5-11 The primary structure floor plan modelled by ANSYS

5.3.3.2 Material Properties

As explained in the previous chapter, this study was conducted within the scope of serviceability analysis. Thus, during the analyses, most of the components deformed within their elastic range. Steel, concrete and glass were defined as key materials used in the finite element models. Details of these materials are listed in Table 5-5.

Table 5-5 Element and material details for the finite element models

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column	$1500 \times 600 \text{ mm}^2$		Linear Elastic	BEAM4
Shear Core	400mm thick	Concrete	$E = 2.5 \times 10^{10} \text{ Pa}$	SHELL63
Shear Wall	300mm thick		$\mu = 0.15$	
Infill Wall	200mm thick	density = 2400 kg/m^3		
Floor Slab				
Façade Panel	10mm thick	Glass	Viscoelastic $G_0 = 2.74 \times 10^{10} \text{ Pa}$ $G_b = 6.05 \times 10^{10} \text{ Pa}$ $1/\beta = 0.53$ density = 2390 kg/m^3	SHELL63
Façade Frame	150UC37	Steel	Linear Elastic $E = 2.0 \times 10^{11} \text{ Pa}$ $\mu = 0.29$ density = 7850 kg/m^3	BEAM4

To simplify the analysis, the following issues were addressed according to the scope of this study:

- The elastic properties of materials were well defined for both steel and concrete;
- The inelastic properties of concrete were not defined for that the analyses were conducted within the elastic range of steel and concrete;
- Visco-elastic properties of glass were defined in detail;
- The reinforcement in concrete components was ignored.

5.3.3.3 Connection Properties

On the basis of the investigation discussed in above sections, connections between structural and non-structural components were the focus of this analysis. Since the non-structural components investigated in this study are partition walls and façades, the bolted connection between the façade system and the primary structure, which is discussed in previous section as composite connections (i.e. connections between different materials), as well as the connections between partition walls and the primary structure were analysed in detail.

The rigidity of these two types of connections is a sensitive factor to evaluate the influence of the connection properties on the contribution of non-structural components to the overall structural performance. Two types of connection rigidity were defined: flexible (pinned connection) and rigid (rigid connection). Following are the detailed definitions of the pinned connection and the rigid connection.

The pinned connection here means only UX, UY, UZ, which are translations in X, Y, Z directions, are constrained. The rotational degrees of freedom (ROTX, ROTY, ROTZ) are totally released;

The rigid connection here means that the total six degrees of freedom (UX, UY, UZ, ROTX, ROTY, ROTZ), both translations and rotations, are constrained.

These two types of connections represent the two extreme scenarios. Thus, by analysing the structures under these situations, the influence of connection properties can also be evaluated.

5.3.3.4 Assumptions, Boundary Conditions and Modelling Approach

Details of components and materials as well as connections are discussed in previous sections. Based on the discussion, a series of finite element models were developed.

To simplify the modelling process, the following assumptions are made in the analysis:

- Only concrete core, shear walls and a wide column are modelled as the primary structure (Figure 5-11). Other structural components such as band beams and columns are ignored in the finite element model;
- The shape of the building is regulated to a rectangular shape to eliminate the complexity of the loading condition;
- The connections between structural components are considered as rigid connections.

In terms of the boundary conditions, as shown in Figure 5-12, the bottom of the building was defined as fixed, to satisfy the initial design of this tall building. A 1KPa equivalent lateral load was applied to each level of the structure, to represent the service load (the 1KPa wind pressure was assumed based on the historic information from Bureau of Meteorology; in the finite element model, this 1KPa pressure was applied to the face of the building where beams and columns were connected).

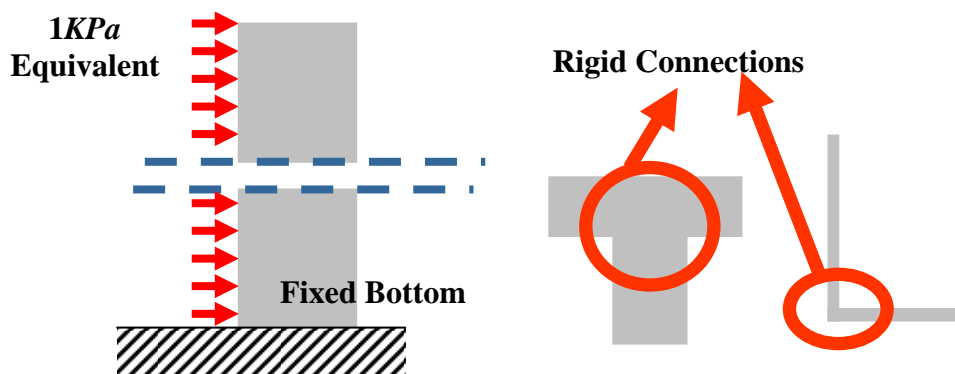


Figure 5-12 Boundary and loading conditions of the finite element models

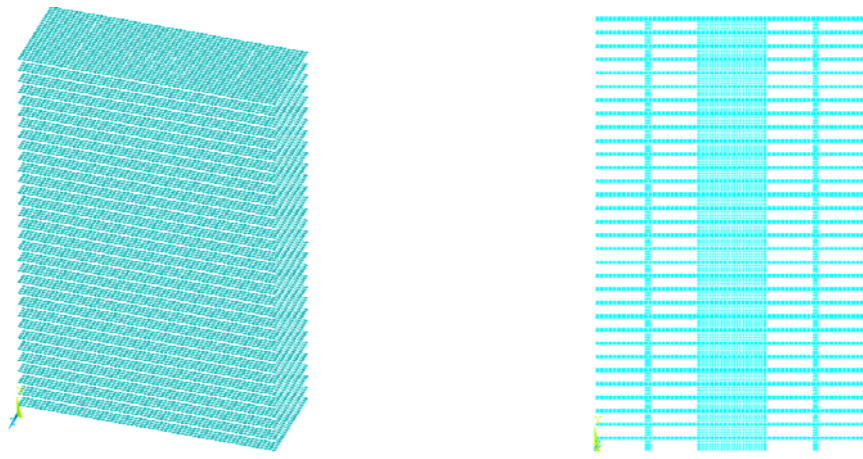
There are three stages in the analysis (Figure 5-13):

- The analysis of the primary structure;
- The analysis of the primary structure with partition walls;
- The analysis of the primary structure with the façade system.

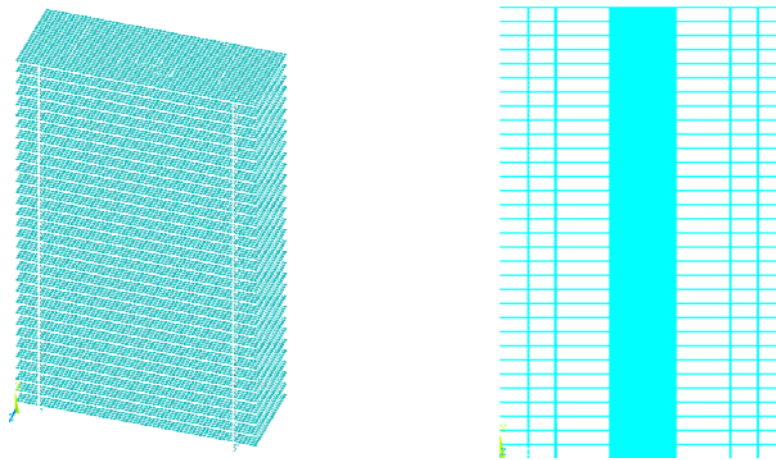
At the second and third stage, following scenarios were detailed:

- Assessment of the influence caused by changing the quantity of non-structural components
 - The analysis of the primary structure with one set of partition walls;
 - The analysis of the primary structure with two sets of partition walls;
 - The analysis of the primary structure with one set of the façade system. Here one set means façades were installed at one storey of the two building faces which are parallel to the loading direction;
 - The analysis of the primary structure with full sets of façade system. Here full sets means façades were installed all along the building height at the two faces which are parallel to the loading direction;
- The assessment of the influence cause by changing properties of connections between structural and non-structural components
 - The analysis of the primary structure with two sets of partition walls fixed to it;
 - The analysis of the primary structure with two sets of partition walls pin-connected to it;
 - The analysis of the primary structure with full sets of façade system fixed to it;
 - The analysis of the primary structure with full sets of façade system pin-connected to it.

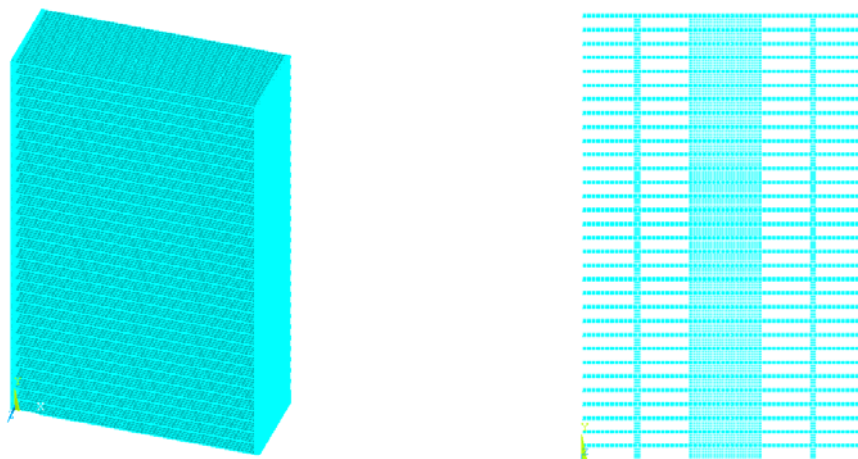
To evaluate the contribution of different non-structural components to the overall structural performance, storey drift of the structure under different scenarios were compared. On the basis of the results obtained from the finite element analysis, the sensitive parameters such as the quantity of non-structural components and the connection properties were also discussed.



(a) Primary structure



(b) Primary structure with partition walls



(c) Primary structure with façade system

Figure 5-13 Finite element models for the three analysing stages

5.3.4 Results and Discussion

By conducting the analyses described in previous sections, some preliminary results are obtained.

Figure 5-14 shows the comparison of storey drifts and inter-storey drifts with and without partition walls. It is clear that by including one set of partition walls in the primary structure, if the walls are rigidly connected to the structural system, the top deflection of the building decreases from 180mm to 163mm and the maximum inter-storey drift ratio can be reduced from 0.2% to 0.18%. Further, as shown in Figure 5-15, when the quantity of partition walls increases from one set to two sets, under the same connection properties, the total storey drift of the structure is further reduced to 145mm and the maximum inter-storey drift ratio is reduced to 0.16%.

The storey drift and inter-storey drift ratio of the structure with and without the façade system are plotted in Figure 5-16. From the graph, it is observed that when including one set façades in the structural analysis, the maximum building deflection is reduced from 180mm to 177mm . Alternatively, if full sets façade are included in the analysis, under the same connection conditions, a maximum 12mm decrease of the storey drift (0.01% decrease of the inter-storey drift ratio) can be achieved, as shown in Figure 5-17, the top deflection of the structure with full sets façade is 168mm .

To assess the effects of changing the quantity of non-structural components on the structural performance, the improvements of storey drift caused by including different amount of non-structural components are quantified, as shown in Figure 5-18 and 5-19. Figure 5-18 shows the influence of the quantity of partition walls on the storey drift of the building. It is clear that by increasing the quantity of partition walls from one set to two sets, the structural stiffness increases, from 9% to 19%. Similar to the partition walls, by increasing the quantity of façades, the stiffness of the building also increases, as shown in Figure 5-19. It is noted that in Figure 5-19, the contribution of full sets façades to the building stiffness below level 9 is smaller than that of one set façades. It can be explained by the P-Delta effect. The excessive gravity loading brought by the full sets façade system is much greater than that from the one set façades. Thus, in the P-Delta analysis, it is not hard to observe that even though the overall stiffness of the structure increases, extra deflection appears, especially at lower levels, until the stiffness contribution balances out the second-order deflection.

The influence caused by changing connection property is another variable investigated in this study. Rigid connections and pin connections were applied to both the partition walls and façades. The storey drifts of structures under different connection conditions are plotted in Figure 5-20 and 5-21. From Figure 5-20, it can tell that when increasing the rigidity of connections between partition walls and the primary structure, the storey drift of the structure decreases accordingly, from 180mm to 163mm, with the maximum inter-storey drift ratio decreases from 0.2% to 0.18%. However, in terms of the façade system, when the connections between the façade system and the primary structure are changed from pinned to rigid connections, only 1mm reduction of the storey drift can be obtained, as shown in Figure 5-21.

Figure 5-22 and 5-23 quantify the stiffening effects of partition walls and façades to the structure under different connection properties. From these two figures, it can identify that by increasing the connection rigidity, the stiffening effect of partition walls increases. As shown in Figure 5-22, when the connections between partition walls and primary structure are changed from the pinned connection to the rigid connection, the contribution of partition walls to the storey drift increases from 17.8% to 19%. However, with the increase of connection rigidity, there is no significant contribution from the façade system to the structural stiffness. As shown in Figure 5-23, only 0.4% increase of the structural stiffness can be achieved.

It is worth mentioning that, by comparing the results obtained from this preliminary finite element analysis with the existing literature (Su et al. 2005), similar conclusions can be found in terms of the stiffness contribution of non-structural components, especially partition walls. This to some extent ascertains the results in this study, and ensures the reliability of the finite element model developed for the case-study building.

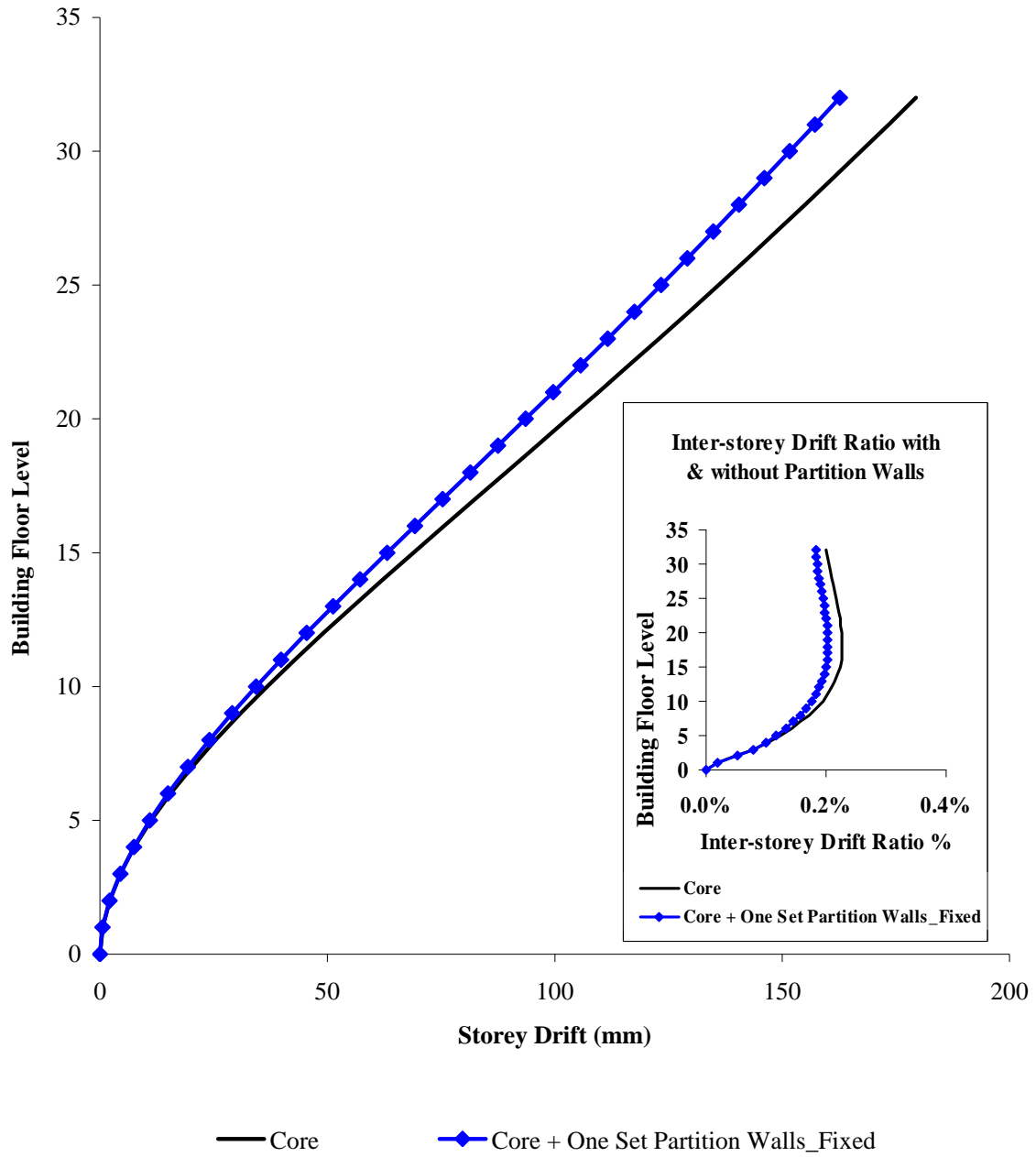


Figure 5-14 Storey drift and inter-storey drift ratio of the building with and without partition walls

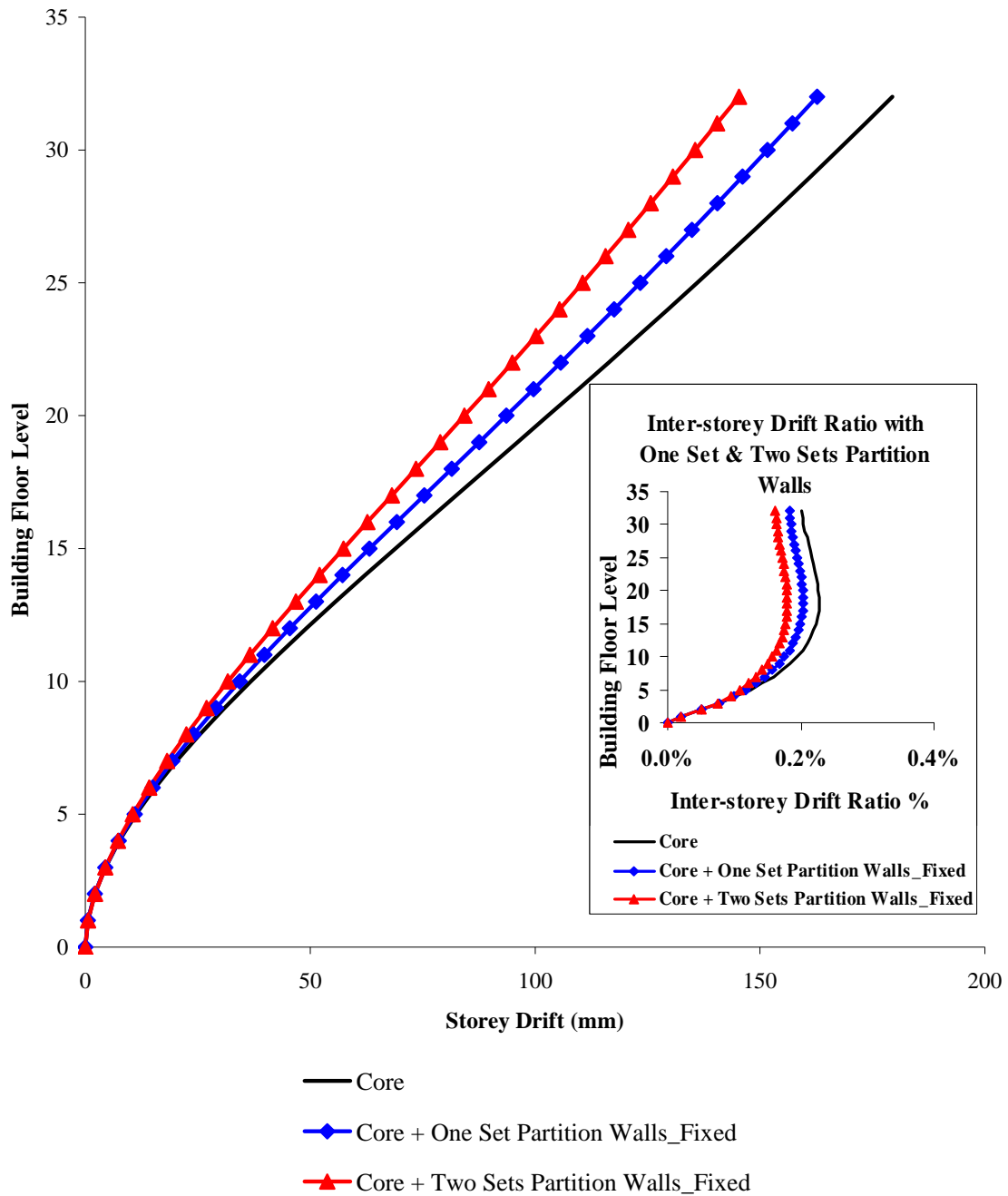


Figure 5-15 Storey drift and inter-storey drift ratio of the building with one set and two sets partition walls

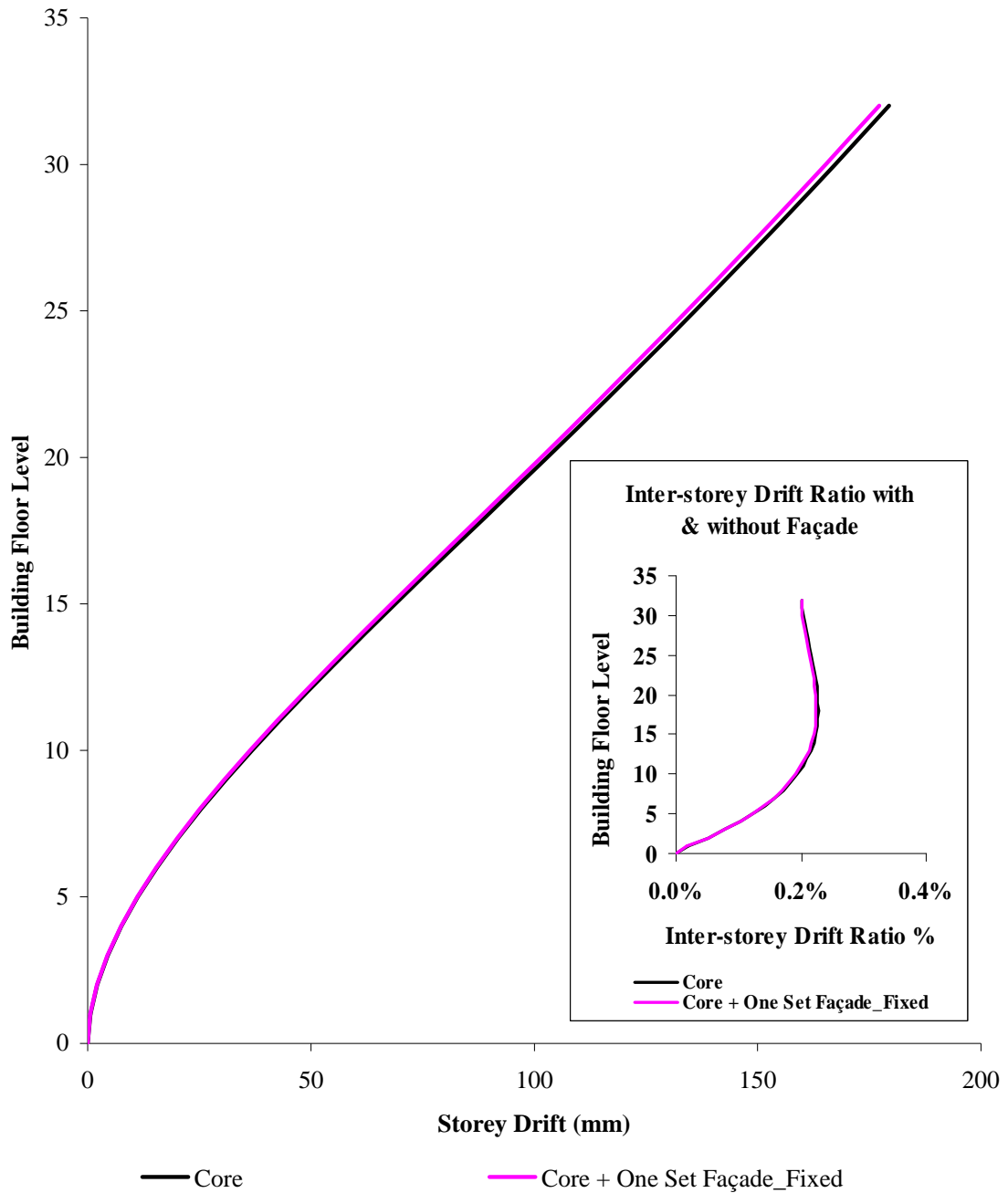


Figure 5-16 Storey drift and inter-storey drift ratio of the building with and without façades

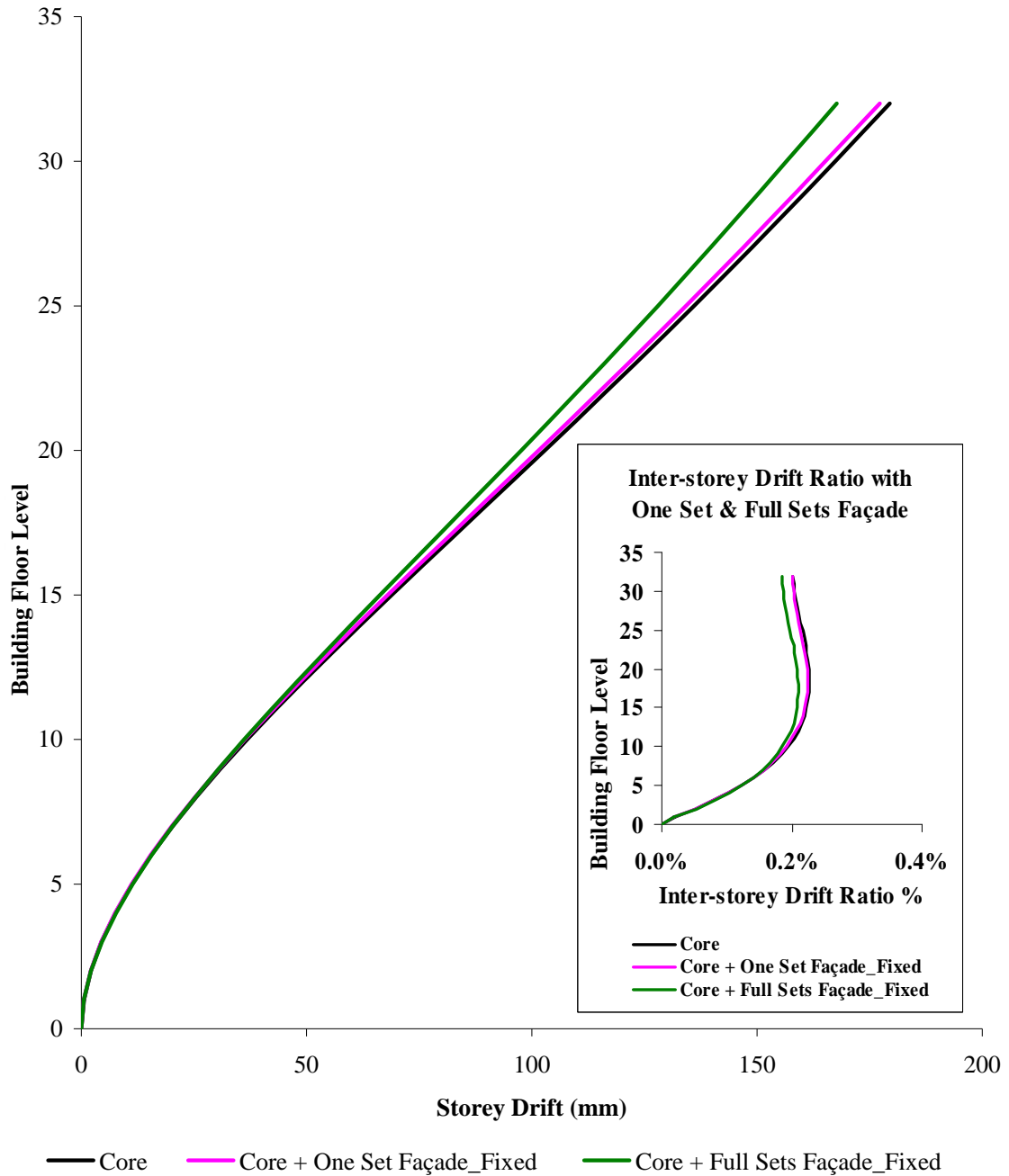


Figure 5-17 Storey drift and inter-storey drift ratio of the building with one set and full sets façade

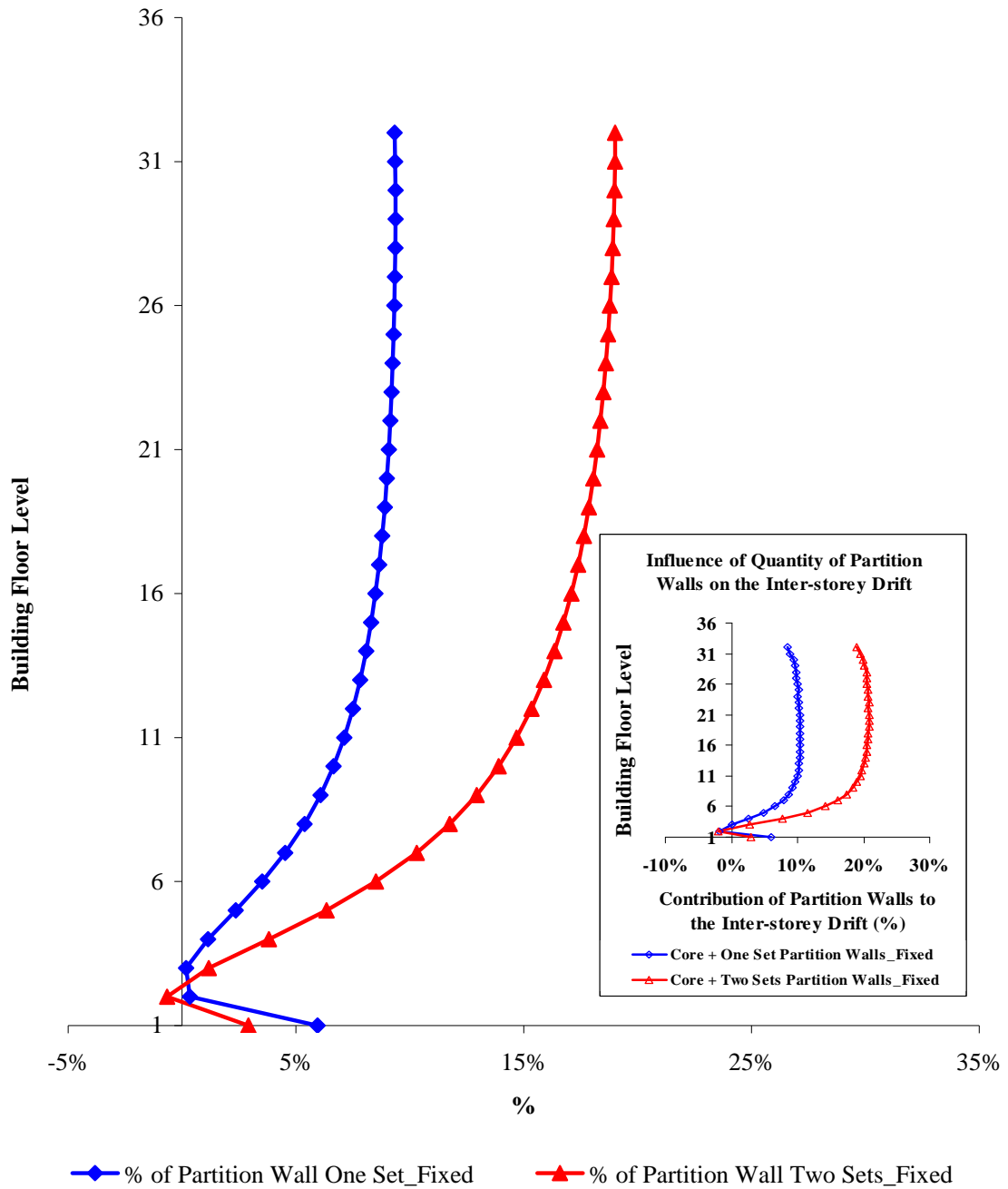


Figure 5-18 Contribution of partition walls with different quantities

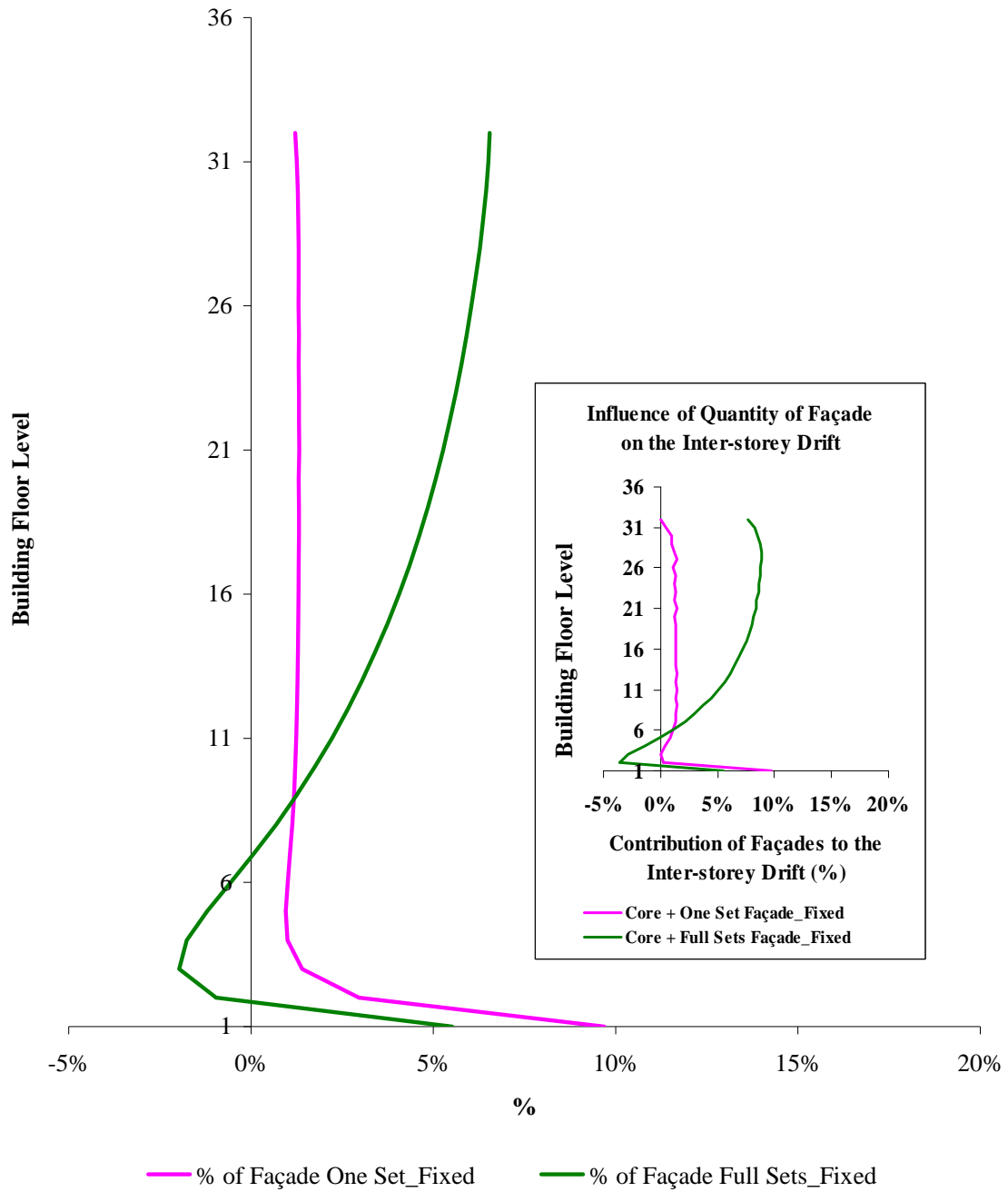


Figure 5-19 Contribution of façades with different quantities

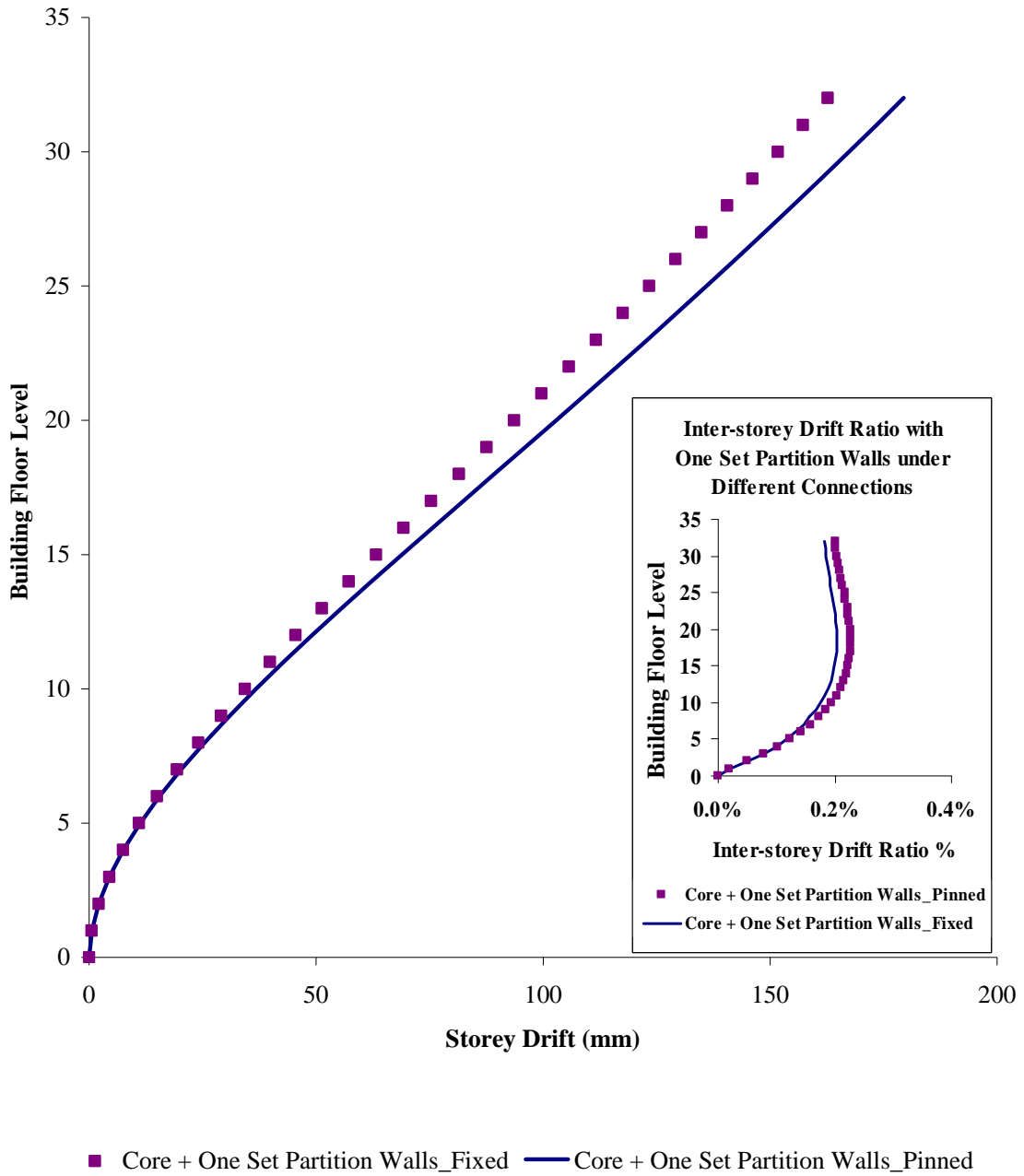


Figure 5-20 Storey drift and inter-storey drift ratio of the structure with partition walls under different connection conditions

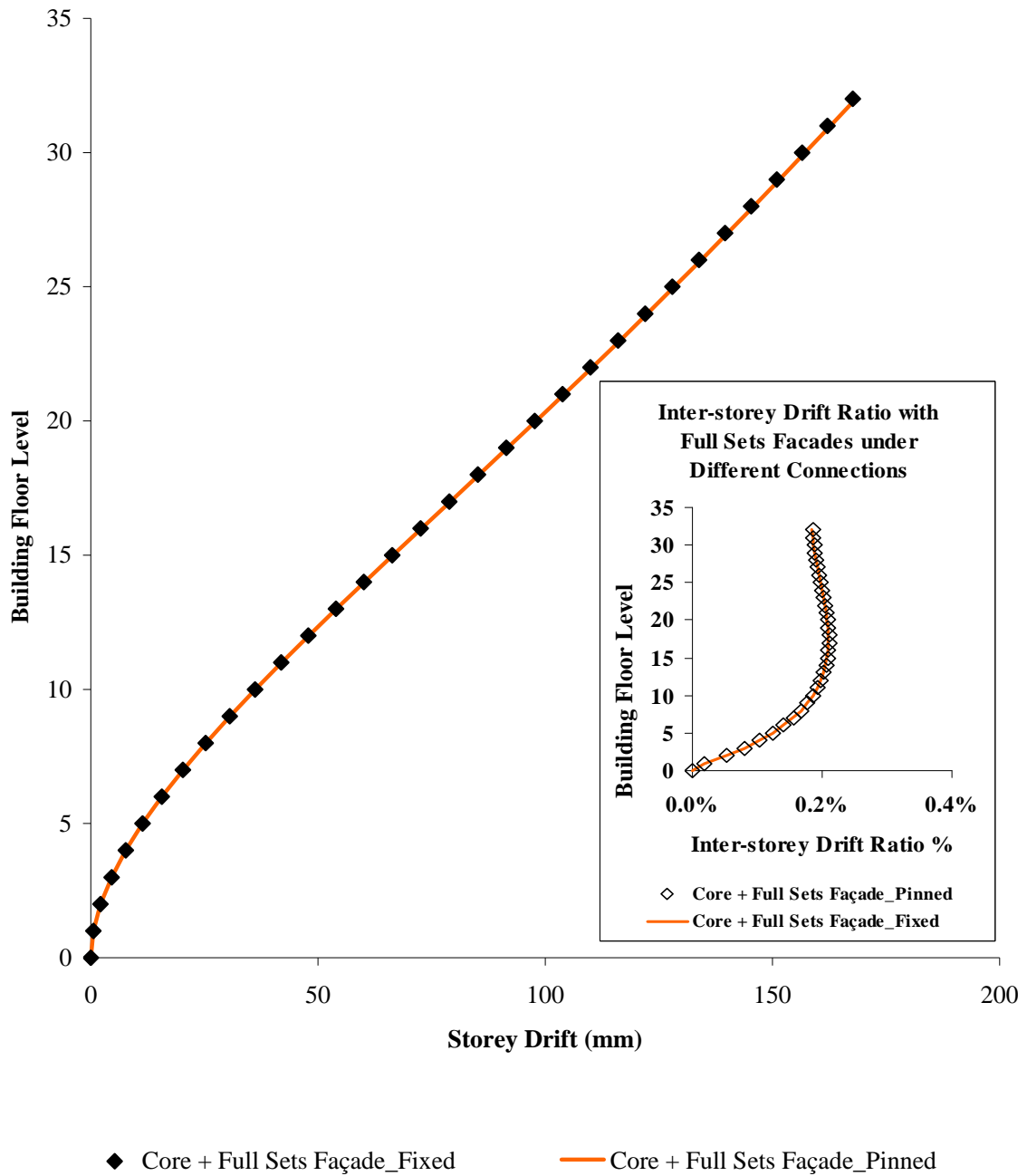


Figure 5-21 Storey drift and inter-storey drift ratio of the structure with full sets façade under different connection conditions

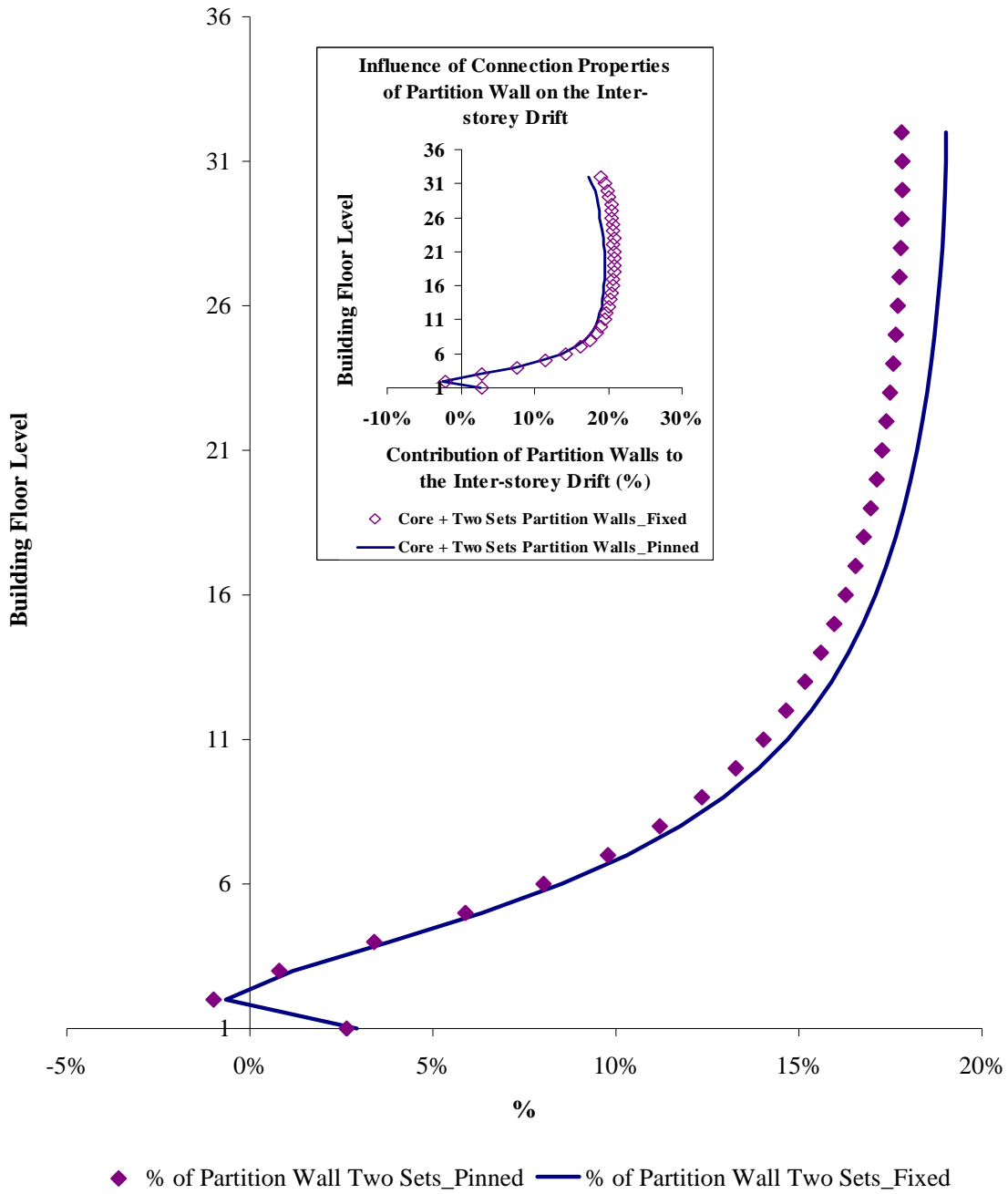


Figure 5-22 Influence of connection properties between partition walls and the primary structure on the storey drift of the building

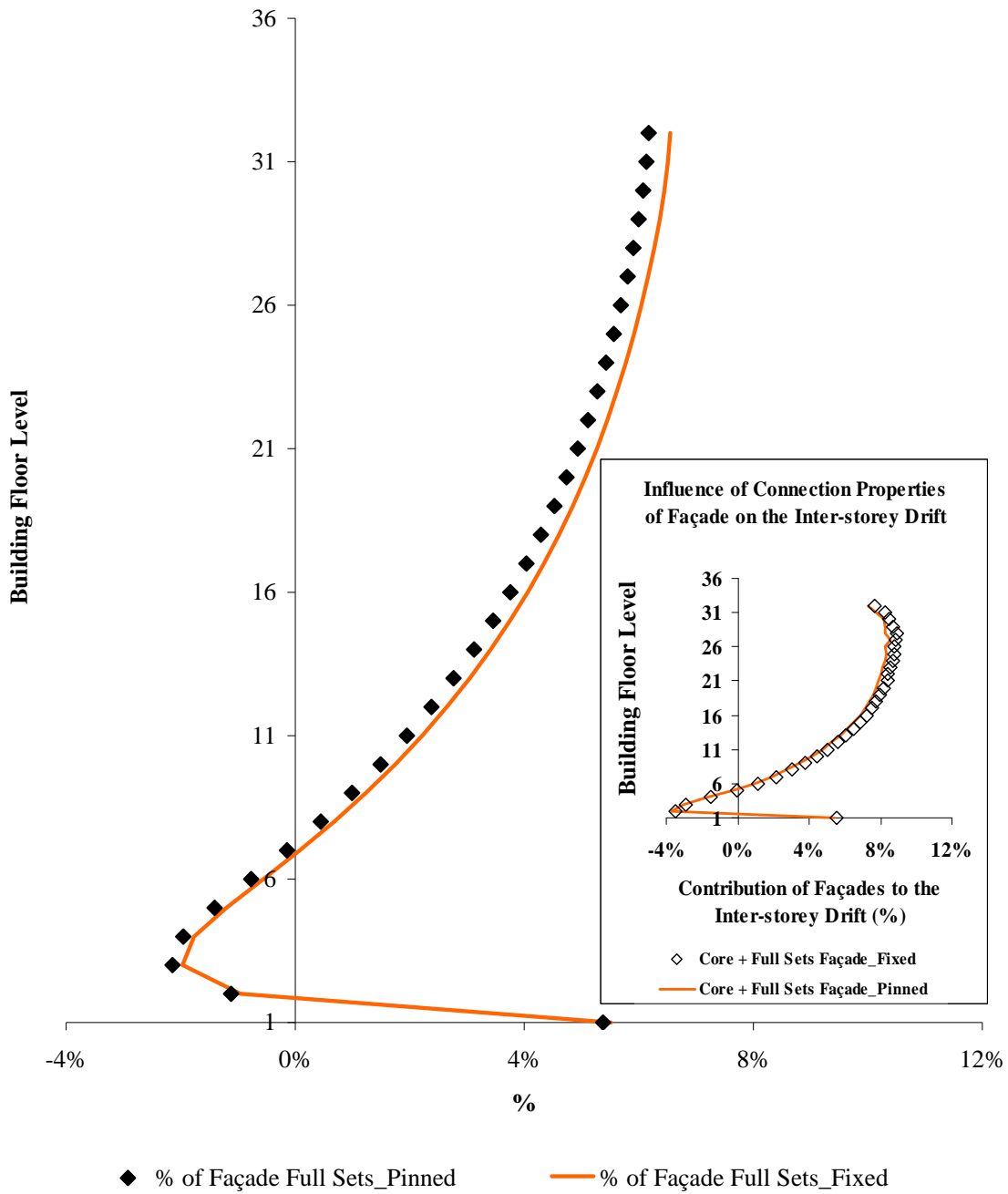


Figure 5-23 Influence of connection properties between façades and the primary structure on the storey drift of the building

5.3.5 Conclusions Based on the Preliminary Finite Element Analysis

Based on the discussion in the above section, it is concluded that by including non-structural components in the structural analyses, the structural stiffness increases. Some key findings are detailed as below.

- By including partition walls and façades in the structural analysis, the stiffness of the structure increases significantly;
- The quantity of non-structural components has influence on the storey drift of the building;
- The storey drift of the building can be affected by the change of connection properties between non-structural components and the primary structure;

In terms of the influence caused by changing the quantity of the non-structural components:

- By increasing the quantity of the partition walls, the structural stiffness increases. In this study, 10% increase of the structural stiffness can be achieved by including an extra set of partition walls in the structural analysis;
- The quantity of façades can also affect their contribution to the structural stiffness significantly. In this study, when the façades are increased from one set to full sets, their contribution to the structural stiffness increases from 2% to 7% accordingly;

In terms of the influence caused by changing the connection rigidity:

- Increasing the rigidity of the connections between partition walls and the primary structure, the contribution of partition walls to the stiffness of the structure increases. In this study, a 3% extra contribution is obtained;
- The change of the storey drift is not obvious when the connections between façades and the primary structure are changed from pinned to rigid. In this study, it is only 0.4% approximately.

Based on the above findings, even though the influencing factors such as connection properties and quantity of non-structural components vary, it is clear that the

contribution of non-structural components to the overall performance of the structure is significant and thus, needs to be carefully considered in the structural analysis.

5.4 Conclusions

A case-study building was investigated and analysed in this study. Field reconnaissance and finite element analyses were carried out focusing on the contribution of non-structural components to the structural performance and the different influencing factors such as the quantity of non-structural components and the rigidity of connections between non-structural components and the primary structure.

In the field investigation of the case-study building, the main design drawings were reviewed, accompanied by site trips and communications with structural engineers. The details of different types of connections and properties of different non-structural components were analysed. From the observation, it is summarised that there are mainly three types of connections in the building: connections between structural components, connections between non-structural components and connections between non-structural components and the primary structure. Under each category, detailed classifications were investigated on the basis of the rigidity of the connections. The connections between structural components are normally considered as rigid connections if no specifications are provided in the design. Details of connections between non-structural components, for example, connections between façade panels and the façade frame, are generally part of the properties detailed in the design of the non-structural components and thus, are standardised specification of different non-structural components. Hence, it is concluded that the most critical factor in this analysis is the connection between non-structural components and the primary structure because that the rigidity of this type of connections directly decides the level of integrity of the building as a system with both structural components and the non-structural components. Consequently, the contribution of non-structural components to the overall structural behaviour is influenced by this type of connections.

On the basis of the field investigation, finite element models were developed to analyse the contribution of different non-structural components to the structural performance. According to the results obtained from the analyses, it is noted that by including different non-structural components in the structural analyses, the structural stiffness changes accordingly. Moreover, the overall structural behaviour is also sensitive to the

quantity of non-structural components and the connection details between non-structural components and the primary structure. In this study, by including only limited number of partition walls in the analysis, the maximum reduction of the building top deflection can reach to 19%. Similarly, the inclusion of the façade system to the structural analysis causes a 7% increase to the building stiffness. Furthermore, it is also revealed that with the increase of the quantity and rigidity of non-structural components, their contribution to the structural stiffness increases accordingly.

The preliminary conclusions drawn from this chapter prove that the contribution of different non-structural components to the building performance should not be ignored during the structural analyses. However, to validate the findings and to investigate the complicated structural behaviour by including different non-structural components in the structural analysis, further laboratory testing and systematic analyses of tall buildings are required. These two parts are to be discussed in Chapter 6 and Chapter 7.

CHAPTER 6

LABORATORY TESTING

6.1 Introduction

The findings from Chapter 5 reveal that based on analysis, the inclusion of non-structural components in a building significantly add to the overall structural stiffness of a building. Moreover, the quantity of non-structural components, as well as properties of connections between non-structural components and the primary structure also affect the contribution of non-structural components to the overall structural performance.

However, details such as influencing parameters and appropriate structural analysing theories still need to be investigated and validated. This chapter presents the testing of a laboratory model to confirm the analytical findings.

The aim of the laboratory testing is to demonstrate the contribution of different non-structural components to the performance of multi-storey buildings, to validate the influencing parameters identified from the preliminary finite element analyses and to obtain a sophisticated understanding of the overall building performance. Limited by the accessibility of resources, a simplified laboratory model with 1:100 scaling factor was developed based on the scaling theory from Sabnis (1983). The design of the model includes the selection of material properties and element dimensions. A 3m timber model was finally developed specifically for this study. Accelerometers were involved in the model testing as the main measuring sensors on the basis of the sensor calibration and selection conducted beforehand (Li et al. 2007).

According to the Australian Standards (AS4063, AS2098 and AS2269), the properties of timber vary significantly among different specimens. Hence, the three-point and four-point tests were conducted to evaluate the elastic properties of the timber specimens used in this study.

Plug tests were carried out in order that the structural motions could be captured under a relatively controllable environment. Factors such as cable arrangement and bottom fixing conditions were investigated to eliminate the testing error. The final set-up of the model testing was proposed by taking into account the above.

There are three stages in the testing program. In the first stage, the structural model was tested without any non-structural panels attached to it. In the second stage, the non-structural panels were connected to the primary structure by double roller connections designed to replicate the stiffness observed in the field reconnaissance (refer Chapter 4). After this, the connections between non-structural panels and the primary structure were changed from double roller connections to rigid connections (this means the panels were tightly fixed to the structural frame). At each stage, the model was tested under different loading grades to obtain the storey drift and the force-displacement relationship of the structure. Consequently, the behaviour of the model with and without non-structural panels was analysed.

Data collected from the laboratory testing were filtered and analysed by using Fast Fourier Transform to convert the acceleration-time history obtained from the laboratory tests to the displacement-time. Storey drifts and force-displacement relationships of the model with different configurations were then plotted and compared.

Finite element models were developed according to the testing results and were adopted for the further prediction of the structural behaviour. A series of analyses were carried out by using the validated finite element model. The compatible results obtained from the laboratory testing and the finite element analyses demonstrates once again the capability of the finite element model to represent the laboratory model thus can be used in further investigation.

6.2 Scaling Theory

Structure models always play an important role in the structural design and analysis. Problems in planning, conducting and interpreting an experimental study of structural behaviour need to be solved by physical modelling (Sabnis 1983). Some case studies from previous researches indicated that the structural model (or reduced-scale model) was able to predict the real structural performance with satisfactory accuracy and much lower costs (Sabnis 1983; Lu et al. 2007). The model test in this study is to facilitate the study to demonstrate the stiffness contribution of different non-structural components, to validate the conclusions obtained from the preliminary finite element analyses, and to evaluate the overall behaviour of tall buildings under lateral loads.

Based on the theory given by Sabnis (1983), the scaling models of structures are capable of representing the real performance of different structures for various purposes. However, “... any structural model must be designed, loaded, and interpreted according to the similitude requirements that relate the model to the prototype structure...” otherwise “an inadequate and even incorrect modelling program” can be led to (Sabnis 1983).

Dimensions and units are the two general but essential aspects in defining and measuring physical quantities. The theory of dimensions can be summarised as the follows (Sabnis 1983):

1. Any mathematical description that describes some aspect of nature must be in a dimensionally homogeneous form.
2. As a consequence of the fact that all governing equations must be dimensionally homogeneous, it can be show that any equation of the form

$$F(X_1, X_2, \dots, X_n) = 0 \quad \mathbf{6-1}$$

Can be expressed in the form

$$G(\pi_1, \pi_2, \dots, \pi_m) = 0 \quad \mathbf{6-2}$$

When the π terms are dimensionless products of the n physical variables (X_1, X_2, \dots, X_n) , and $m=n-r$, where r is the number of fundamental dimensions that are involved in the physical variables.

Regarding the static elastic models, the similitude requirements are summarised in Table 6-1.

6.3 Model Design

6.3.1 Objectives of the Laboratory Testing

A laboratory model specific for this study was developed according to the scaling theory introduced in the above section. The aim of the model testing is to identify the contribution of different components to the structural behaviour. The main objectives of the laboratory testing are:

- To identify a simplified physical model capable of representing the performance of tall buildings;
- To demonstrate the influence of non-structural elements to the overall building performance. It can be subdivided into:
 - Stiffness contribution of non-structural components;
 - Influence caused by different types of connections between the primary structure and the non-structural elements to the overall building performance.

Table 6-1 Similitude requirements for static elastic modelling (Sabnis 1983)

Quantities	Dimensions	Scale Factors
Material related properties		
Stress	FL^{-2}	S_E
Modulus of elasticity	FL^{-2}	S_E
Poisson's Ratio	--	I
Mass density	FL^{-3}	S_E/S_l
Strain	--	I
Geometry		
Linear dimension	L	S_l
Linear displacement	L	S_l
Angular displacement	--	I
Area	L^2	S_l^2
Moment of inertia	L^4	S_l^4
Loading		
Concentrated Load Q	F	$S_E S_l^2$
Line load w	FL^{-1}	$S_E S_l$
Pressure or uniformly distributed load q	FL^{-2}	S_E
Moment M or torque T	FL	$S_E S_l^3$
Shear force V	F	$S_E S_l^2$

6.3.2 Model Design

The dimensions of the laboratory model are decided based on the information collected from the field reconnaissance. It is a common practice that the base to height ratio of tall buildings is from 1:3 to 1:10. Limited by the testing environment such as the floor to roof height of the laboratory, two typical constraints in the laboratory testing were considered:

- a) The height of the model. Since this model needs to be assembled and tested in the structural laboratory, the height of the model should not exceed the practical limits so that it can be accommodated by the laboratory;

- b) The base/roof area of the model. Considering that the measuring sensors need to be installed on various locations of the model, including the roof, certain amount of roof area is required to be secured.

A base-to-height ratio of 1:6 was chosen for the model in accordance with that of the Eureka Tower, the tallest residential building in Melbourne. The base dimensions and the height of this model are finally decided as: 500mm by 500mm by 3000mm.

6.3.2.1 The Selection of Structural Elements and the Material

To optimise the design of the laboratory model, including the element dimensions and material properties, feasibility analyses were carried out by using commercial software package ANSYS10.0.

Following are the assumptions made during the material and element selection process.

- Decided by the repeatability of the test itself, as well as the purpose and scope of this study, the model should be designed to be used for elastic analysis only;
- When scaling the model according to the scaling theory, the reinforcement in the prototype model can be ignored;
- It is assumed that all the connections between structural elements, i.e. beam to column connections, are rigid connections;
- Constrained by the sensitivity of the devices, the stiffness of the model should be controlled.

A simplified frame with shear core model was developed according to the basic dimension requirements of the model design. A three-node beam element BEAM4 and a four-node shell element SHELL63 in ANSYS were used to model beams and columns, and shear walls respectively. Three different options for the selection of the material and elements were proposed. The aluminium and timber are involved as two different types of construction materials. Detailed material properties and element dimensions are presented in Table 6-2, 6-3 and 6-4, for three different design options respectively. The details of the proposed element cross sections of the designed models are shown in Figure 6-1.

The force-displacement relationships of the static lateral behaviour of model option 1 and 2 are plotted in Figure 6-2. It is clear that when subject to the same $3 \times 10^4 N$ lateral load, the top deflections of model option 1 (aluminium model) and 2 (timber model) are $30mm$ and $85mm$ respectively because of the much higher stiffness of model option 1 ($1000N/mm$) than model option 2 ($353N/mm$). It demonstrates that timber is more proper for this specific model testing.

The loading condition is another important factor considered in the model design. According to the weather information from Australia Bureau of Meteorology, the historical maximum wind speed in Melbourne CBD area from February 2006 to February 2007 is around $50km/h$, which means the maximum wind pressure is around $125Pa$ (unfactored). For a $50m$ by $50m$ by $300m$ tall building, the design wind pressure can be roughly computed according to AS1170.2:2002 (2002) as from $1.0KPa$ to more than $2KPa$. If consider the wind load to this reduced-scale model, the above information can be roughly converted to a $100N$ lateral load (at service level) applied to the surface of the model. It indirectly indicates that the model option 2 also lacks flexibility in responding to the normal daily wind load.

After reducing the element dimensions of model option 2, measurable top deflection ($11mm$) of the new model option (option 3) was achieved under the loading condition of $60N$ (Table 6-5). Details of this model (option 3) are listed in Table 6-4.

Table 6-2 Material and element dimension summary of the primary structural design of the laboratory model (option 1)

	Material	Type	Qua.	Dimension				Mass/length kg	Total Length m	Usage
				W (mm)	D (mm)	t (mm)	H (mm)			
Frame	Aluminum Beam	I-beam	1	75	100	4.8/6.4	3000	25.051	3	Core (Opt1)
	Aluminum Tube	Square	1	80	80	6	3000	29.082	3	Core (Opt2)
	Aluminum Angle	L-angle	8	50	50	6	3000	15.012	24	Columns
		L-angle	24	40	40	4	500	5.33	12	Beams
		L-angle	24	40	40	4	210	5.33	5.04	Beams
Wall	Timber sheets		40	250	60	3~6				Outside walls
	Timber sheets		40	250	60	3				Infill walls
Other	Brackets		72~100							
	Mass units		24							

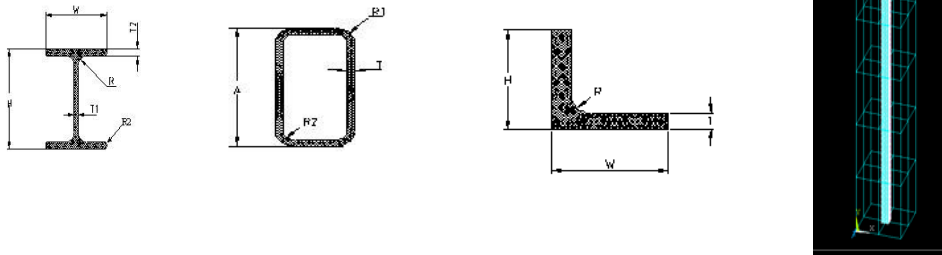


Figure 6-1 Details of the designed model and the element sections

Table 6-3 Material and element dimension summary of the primary structural design of the laboratory model (option 2)

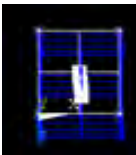
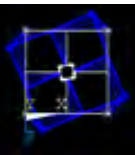

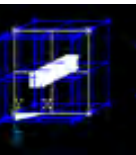
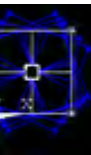
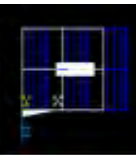

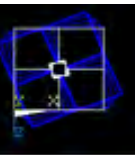
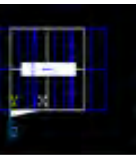

	Material	Type	Qua.	Dimension				Mass/length <i>kg</i>	Total Length <i>m</i>	Usage
				B (<i>mm</i>)	D (<i>mm</i>)	t (<i>mm</i>)	H (<i>mm</i>)			
Frame	Timber beam		4	80	80	3	3000	12	Core	
	Timber beam	Rectangular	12	35	90		3000	36	Columns	
	Timber beam	Rectangular	60	35	75		250	15	Beams	
Wall	Timber plate		40	250	60	3			Outside walls	
	Timber plate		40	250	60	3			Infill walls	
Other	Brackets/ screws		72~100							
	Mass units	Lead blocks	24						Added Mass	

Table 6-4 Material and Element dimension Summary of the Primary Structural Design of the Laboratory Model (option 3)

	Material	Type	Qua.	Dimension				Mass/length <i>kg</i>	Total Length <i>m</i>	Usage
				B (<i>mm</i>)	D (<i>mm</i>)	t (<i>mm</i>)	H (<i>mm</i>)			
Frame	Timber plate		4	80	80	3~6	3000	12	Core	
	Timber beam	Rectangular	4	20	20		3000	12	Columns	
	Timber beam	Rectangular	24	10	10		500	12	Beams	
	Timber beam	Rectangular	24	10	10		80	19.2	Beams	
	Timber beam	Rectangular	24	10	10		210	5.04	Beams	
Wall	Timber plate		40	250	60	3~6			Outside walls	
	Timber plate		40	250	60	3~6			Infill walls	
Other	Brackets/ screws		72~100							
	Mass units	Lead blocks	24						Added Mass	

Table 6-5 Modal analysis of the designed model options

Set	Time/Frequency	
	Option 1	Option 2
1	0.03	0.59
2	0.05	0.74
3	0.11	1.16
4	0.11	1.92
5	0.14	2.64

Mode	1	2	3	4	5
Option 1					
Frequency	0.03	0.05	0.11	0.11	0.14
Option 2					
Frequency	0.59	0.74	1.16	1.92	2.64

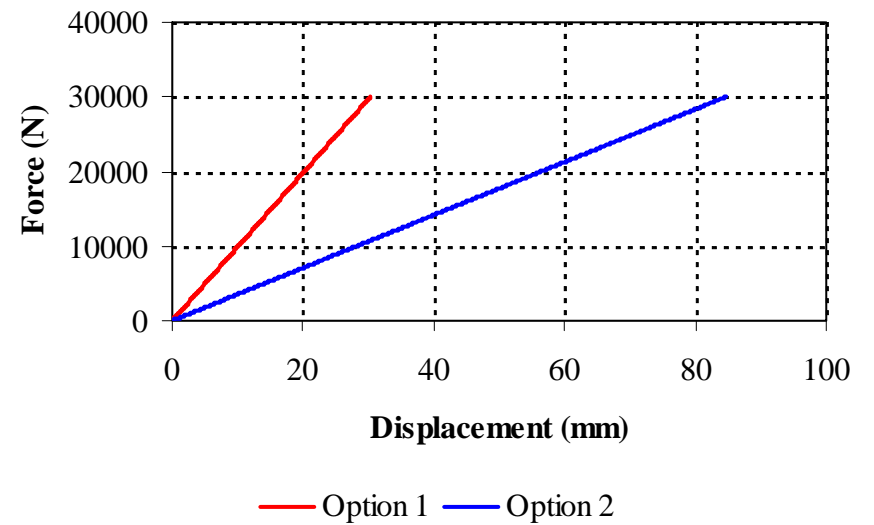


Figure 6-2 Comparison of F-D relationship of designed model options 1 and 2

Table 6-6 Scaling factor analysis_ static and dynamic

SIMILITUDE SCALE FACTORS FOR THE TEST MODEL			Mode	1	2	3	4	5		
Parameter	Relationship	Model/prototype								
Length	S_l	0.01	Timber without extra mass							
Young's modulus	S_E	0.68		Frequency	0.04	0.04	0.10	0.13	0.16	
Stress	$S_\sigma = S_E$	0.68		Timber with 40kg extra mass						
Strain	S_σ / S_E	1.00			Frequency	0.03	0.03	0.07	0.09	0.11
Density	$S_\sigma / (S_a S_l)$	0.227								
Force	$S_\sigma \times S_l^2$	6.8×10^5								
Frequency	$S_l^{-0.5} \times S_a^{0.5}$	173.1								
Mass	$S_\sigma / S_a \times S_l^2$	2.27×10^7								
Acceleration	S_a	299.6								

Reduced-Scale Model		Full-Scale Model		Dis. Scaling m	Difference
Dis. $\times 10^{-3} m$	Force N	Dis m	Force N		
0	0	0	0	0	0.0%
1.17	6	0.12	88235.3	0.12	0.4%
2.34	12	0.23	176470.6	0.23	0.4%
3.51	18	0.35	264705.9	0.35	0.4%
4.68	24	0.47	352941.2	0.47	0.4%
5.86	30	0.58	441176.5	0.59	0.4%
7.03	36	0.70	529411.8	0.70	0.4%
8.20	42	0.82	617647.1	0.82	0.4%
9.37	48	0.93	705882.4	0.94	0.4%
10.54	54	1.05	794117.7	1.05	0.4%
11.71	60	1.17	882353	1.17	0.4%

Static F-D Relations of Timber Model with Smaller Element Cross Sections

Displacement (mm)	Force (N)
0	0
3	15
6	30
9	45
12	60

6.3.2.2 Scaling Factor Analysis

According to the scaling theory explained in previous section, when the dimensional scaling factor S_l is 1:100, other scaling factors can be calculated based on the similitude requirements for static elastic modelling (Table 6-6).

Consequently, material properties and element details for both the reduced-scale model and the full-scale model are presented in Table 6-6.

Comparing the results from both theoretical analysis and finite element analysis, only 0.4% difference can be identified. It thus validates the scaling factors involved in the model development stage.

6.3.2.3 Miscellaneous

Besides the major structural factors analysed above, following issues are also addressed in the model development.

- **Connection properties**

Theoretically, the connections between beams and columns are considered as rigid. Liquid nails and glue for timber-to-timber connections were used as the basic bonding. The triangular joint enhancement was also adopted to ensure the rigidity of the structural connections (Figure 6-3).



Figure 6-3 Connection details of the laboratory model

There are two types of connections designed for the infill panels to the structural frame: rigid and rotation-flexible. The rigid connection was achieved by tightly fixing/inserting the infill panels to the structural frame. The rotation-flexible connection was the connection using double roller to attach the panels to the frame (Figure 6-4).



Figure 6-4 Details of the double roller connection of infill panels to the frame

- **Foundation/Base**

Since a tall building is always considered as a vertical cantilever in the theoretical analysis, to constrain the base of the model so that it can be totally fixed to the ground, the model was embedded into a thick hardwood plate which was well bolted to the laboratory floor (Figure 6-5).



Figure 6-5 Bottom fixing details of the laboratory model

6.3.3 Test of Material Properties

Although timber is the widely adopted construction material in Australia, the variety of its properties is still a big concern when accuracy and details are highly demanded in the structural experiment.

According to the requirements of this study, basic elastic properties of timber, such as the elastic modulus under bending and shear modulus need to be clearly defined.

Based on AS/NZS 4063:1992 (1992), AS/NZS 2098:2006 (2006), and AS/NZS 2269:2004 (2004), the standard three-point and four-point tests were conducted to obtain the elastic properties of the timber used in the model construction.

Details of the testing arrangements are shown in Figure 6-6 and 6-7. Seven specimens with a length of 400mm and a cross section of 20mm by 20mm were tested. Each specimen was tested exactly in the same way shown in Figure 6-8 and 6-9.

The testing machine used for both three-point and four-point test is Instron 1122, which has a 5kN loading capacity. It also has the function of controlling the deflecting rate of the specimen.

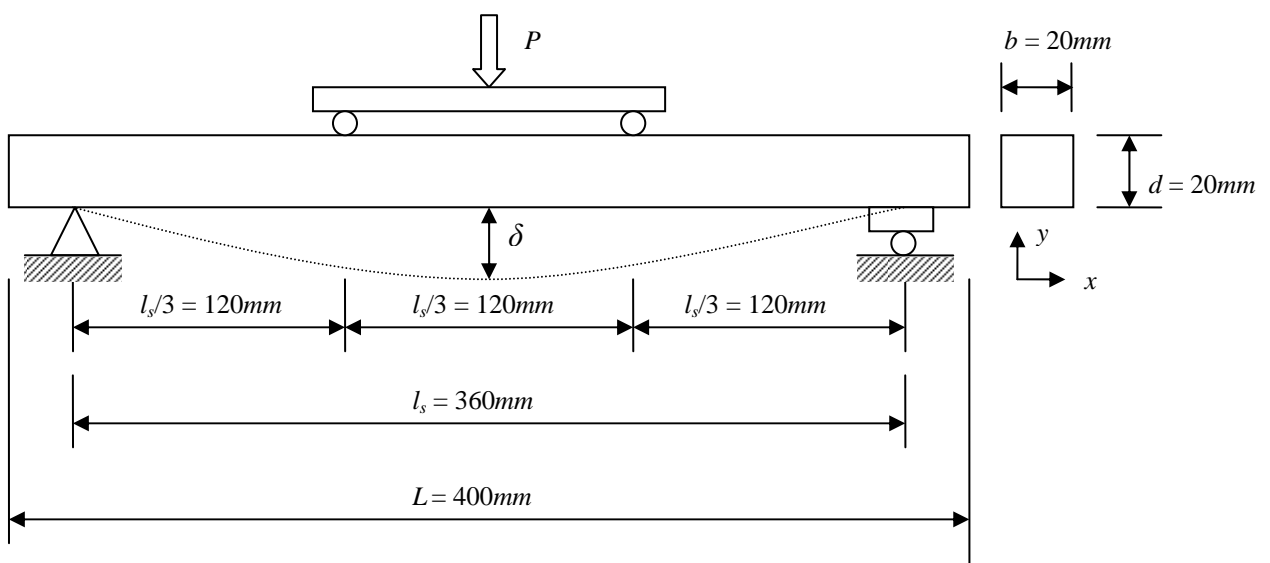


Figure 6-6 Details of four-point test

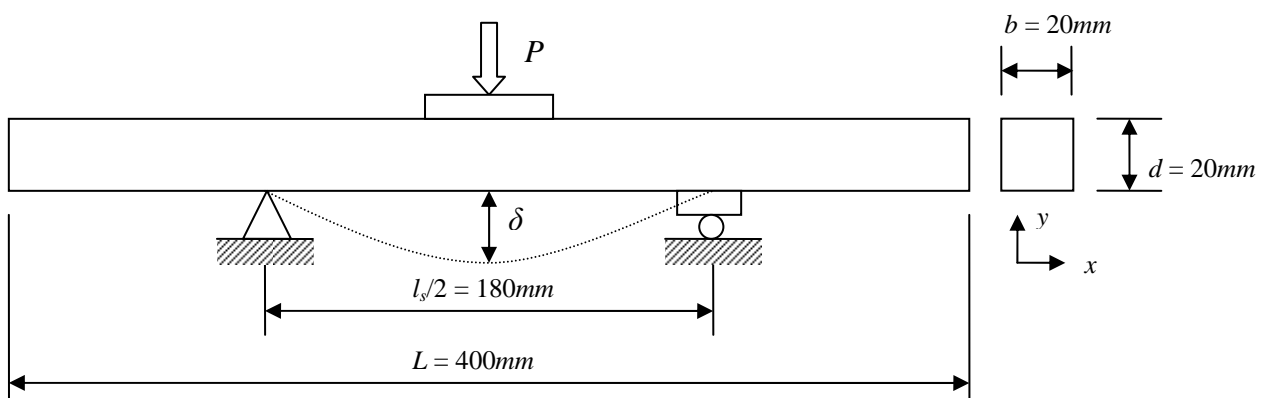


Figure 6-7 Details of three-point test



Figure 6-8 Four-point test



Figure 6-9 Three-point test

Equations 6-3 and 6-4 were used to evaluate the elastic modulus under bending and the shear modulus:

$$E_{true} = \frac{111}{216} \frac{l_s^3}{bd^3} \frac{S_c S_b}{(3S_b - 4S_c)} \quad 6-3$$

$$G = \frac{22.20l_s}{bd} \frac{S_c S_b}{(184S_c - 27S_b)} \quad 6-4$$

Finite element models (Figure 6-10 and 6-11) were also developed to verify the results from the laboratory tests and to demonstrate the settings of the material properties in the model testing. One specimen was used to test the failure mode to ensure the applied load can be well adjusted. The results from the rest 6 specimens are presented in Figure 6-12 and 6-13.

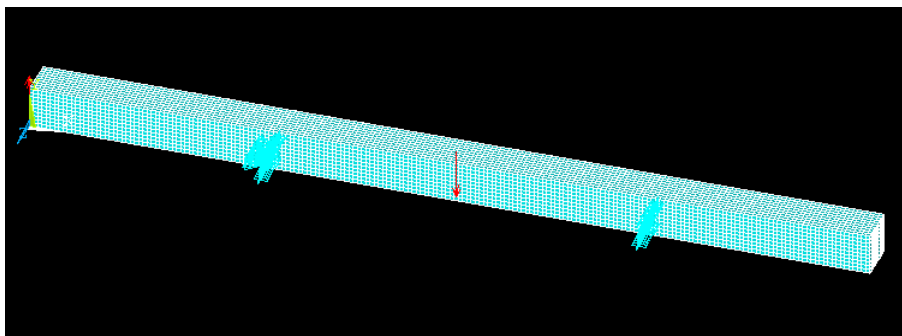


Figure 6-10 Finite element model for the three-point test

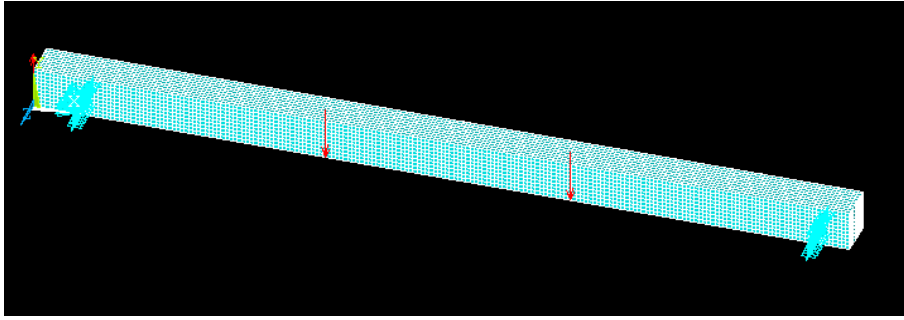


Figure 6-11 Finite element model for the four-point test

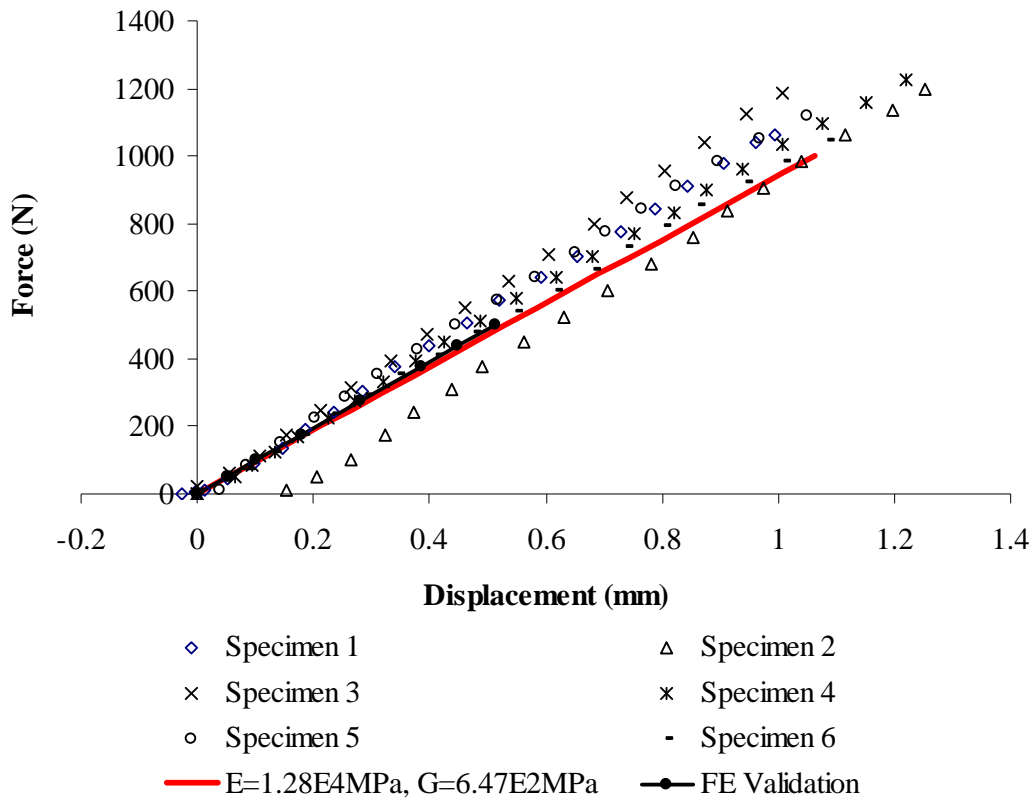


Figure 6-12 Result of the three-point test (FE-Finite Element)

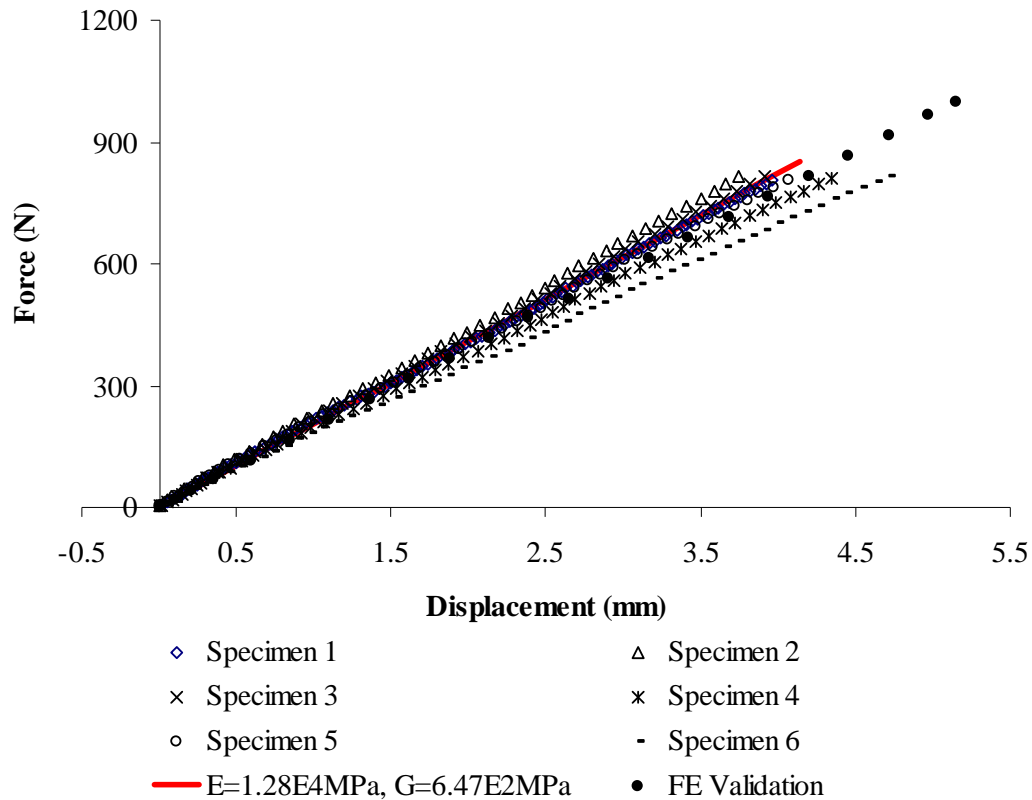


Figure 6-13 Result of the four-point test (FE-Finite Element)

As show in Figure 6-12 and 6-13, the elastic modulus and shear modulus of timber are calculated. Details of the specimen and the calculated elastic properties based on the testing results are listed in Table 6-7. These results can be used as a rough indicator to develop finite element models for the laboratory model. However, detailed calibration is also required by the completion of the laboratory testing.

Table 6-7 Details of the specimen and the testing results

Specimen No	1	2	3	4	5	6	Average
<i>a</i> (mm)	120.0	120.0	120.0	120.0	120.0	120.0	120.0
<i>l_s</i> (mm)	360.0	360.0	360.0	360.0	360.0	360.0	360.0
<i>b</i> (mm)	20.0	20.0	20.0	20.0	20.0	20.0	20.0
<i>d</i> (mm)	20.0	20.0	20.0	20.0	20.0	20.0	20.0
<i>S_a</i>	199.1	211.9	203.9	183.5	194.2	170.9	193.9
<i>S_b</i>	1077.2	1095.1	1183.0	1020.1	1095.3	979.2	1075.0
<i>E</i> (Mpa)	13196.6	14263.8	13222.9	12058.7	12706.9	11128.0	12762.8
<i>G</i> (Mpa)	567.7	492.2	864.5	601.0	689.3	667.1	647.0

6.3.4 Model Instrumentation

A comprehensive study was carried out before the laboratory testing, to select and calibrate potential sensors for this experiment (Li et al. 2007). Details of the sensor selection and calibration are provided in Chapter 3.

In the laboratory testing, considering that the vibration response of the model to the lateral loads is much higher than that of a real building, the requirement of the sensitivity of accelerometers becomes lower than that of the real building measurement.

Details of the measurement devices used in the model testing are as follows.

- Accelerometers: six Dytran 3192A accelerometers were installed to the model, with one at each level (Figure 6-14). Key features of the accelerometers can refer to the specification sheet of accelerometer 3192A discussed in Chapter 3;
- Accessories: main accessories include a laboratory computer with data acquisition card, two signal conditioners, 15m~30m accelerometer cables for each of the accelerometer, cables for signal conditioners, the mounting bases, and the power supply;
- Plug test set-up: the plug test was set up by using a crown block, a string and the weights (Figure 6-15). The amount of weights controls the applied transient load to the model.



Figure 6-14 Details of the laboratory model set-up



Figure 6-15 Details of the plug test set-up

6.4 Model Calibration

The in-house plug test aims to evaluate the performance of the structure and to validate the conclusions obtained from the preliminary finite element analysis.

The set-up of the experimental system is shown in Figure 6-14 and 6-15, as discussed in the previous section. The skeleton frame (with floors while without roof) was tested under several different configurations in order that the influence of different factors, such as the cable arrangement, the bottom fixing conditions, can be carefully addressed during the result analysis of the model test.

6.4.1 Influence of Cables

To evaluate the influence of cables, following tests were conducted to the laboratory model:

- Cables were arranged along the loading direction (x-axis). Cables were attached to the model. Six accelerometers were attached to the central core of the model. The cables were regularly arranged along the loading direction and were fully attached to the beams and columns of the frame (Figure 6-16)



Figure 6-16 Cables of accelerometers are attached to the frame along the loading direction

- Cables were arranged across the loading direction (z- axis). Cables were attached to the model. Six accelerometers were attached to the central core of the model. The cables were regularly arranged across the loading direction and fully attached to the beams and columns of the frame (Figure 6-17)



Figure 6-17 Cables of accelerometers are attached to the frame across the loading direction

- Cables were hung up to a freestanding steel post. Six accelerometers were attached to the left-front corners of the frame (Figure 6-18) and another six accelerometers (for balancing purpose only) were attached to the right-rear corners of the frame at each level.



Figure 6-18 Cables of accelerometers were hung up to a freestanding steel post

For each test, there are total 7 loading grades, 1.6N, 3.5N, 6.5N, 11.4N, 21.3N, 31.1N and 41.0N. Within each group, the test was repeated 3 times, so that the operational error can be eliminated as much as possible.

Result Analysis and Discussion

The comparison of the influence of cable arrangement on the deflection of the model is shown in Figure 6-19. From the figure, it can tell that the testing results vary according to the different arrangements of the cables. When under 1.6N lateral load, the top deflections of the model are from 0.3mm (cables hung to the freestanding cantilever post) to 0.5mm (cables attached to the model along the loading direction) whilst the finite element result is 0.4mm. Similarly, under 3.5N and 6.5N, the model has larger deflections with the cables being attached to the model along the loading direction. Compared with the finite element analysis results, it is noted that when the cables are hung up to the freestanding cantilever post, the lateral performance of the model is most identical to that from the finite element model.

It indicates that the influence of cables on the lateral performance of the model is significant. The possible reason for the enormous effect of cables is that the model is ultra light-weight, thus the cable mass and the possible stiffness enhancement effect of the cables can not be overlooked in this particular case.

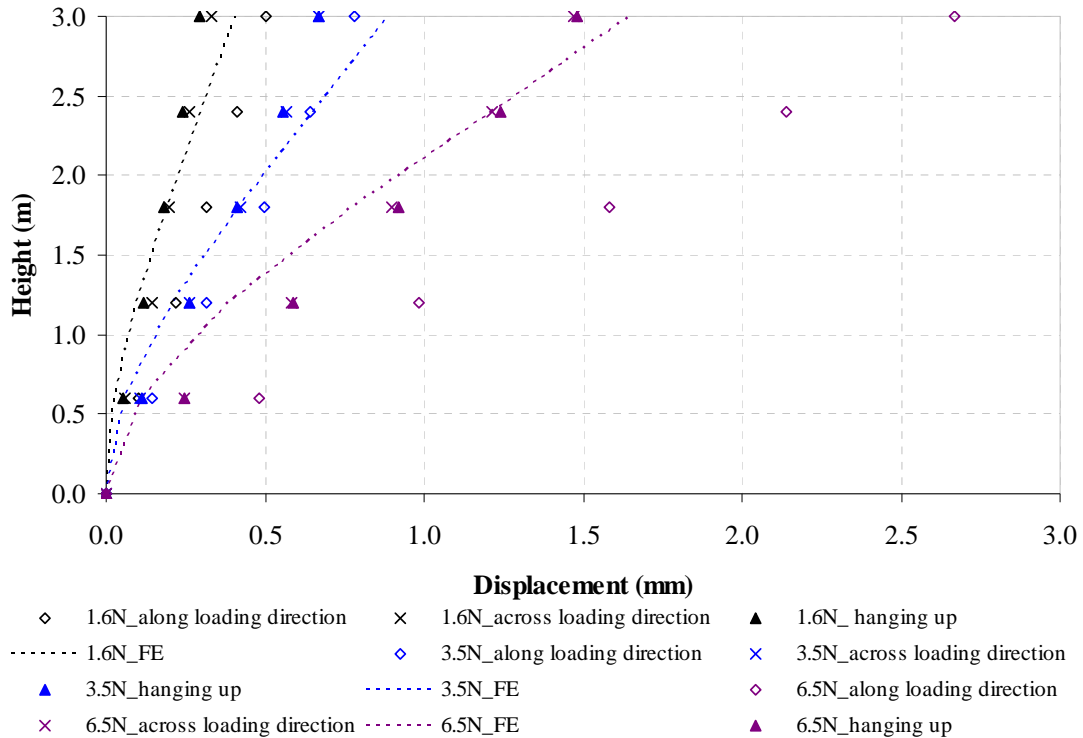


Figure 6-19 Variations of storey drift caused by cable arrangement (FE-Finite Element)

Further analysis of the results indicates that the behaviour of the laboratory model and the finite element model are different at the bottom levels. The finite element model shows typical cantilever behaviour with all the 6 degrees of freedom constrained at the bottom. However, the results of the laboratory model reveal more linear characteristics, from which a certain amount of displacements and rotations are detected. This indicates that further calibration of the model is necessary.

The comparison of the storey drifts obtained from the laboratory model testing with accelerometer cables hanging up to the freestanding steel post and the corresponding finite element model under different loading grades is shown in Figure 6-20. From this figure, it is clear that the results obtained from both laboratory tests and the finite element analyses are close to each other, except at the bottom part of the model.

The force-displacement relationships obtained from both hanging cable tests and the finite element model are plotted in Figure 6-21. In the chart, the results from both

analyses are close to each other. This validates that hanging cables to an isolated freestanding steel post can minimise the influence of cables.

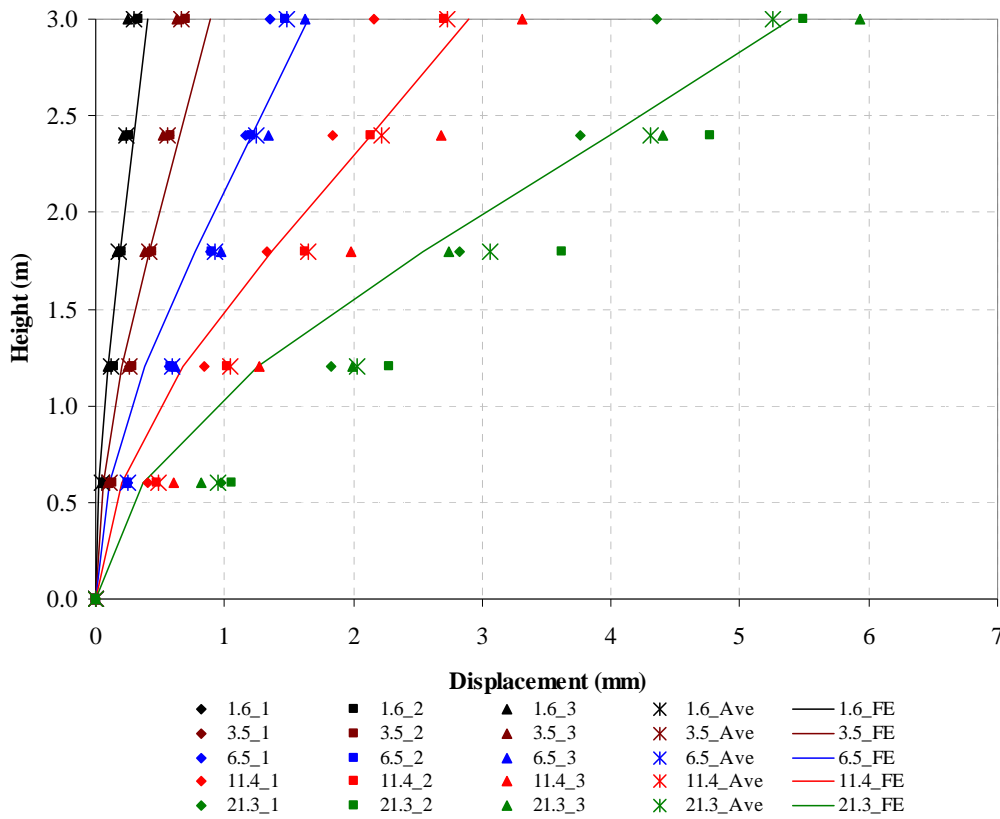


Figure 6-20 Storey drift of the model by hanging the cables to the freestanding steel post (FE-Finite Element)

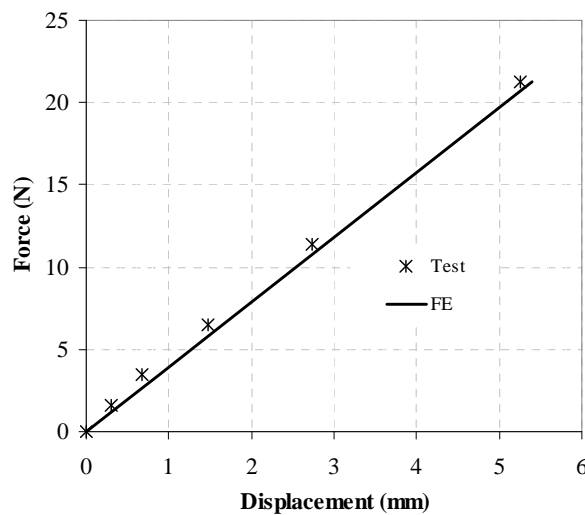


Figure 6-21 Force-displacement relationships of the hanging cable test and the finite element analysis (FE-Finite Element)

6.4.2 Bottom fixing conditions

From the results shown in Figure 6-19 and 6-20, behaviours of the laboratory model and the finite element model are different at the bottom level. The linearity of the storey drift curve obtained from the laboratory model testing illustrates that rotations and translational displacements still exist. Equal angles were used to enhance the bottom rigidity of the laboratory model. However, little improvement was observed from the testing results. Under this circumstance, the adjustment of the boundary conditions of the finite element model is necessary.

Different bottom fixing conditions of the model in the calibration of the finite element model are plotted in Figure 6-22 (a) and (b).

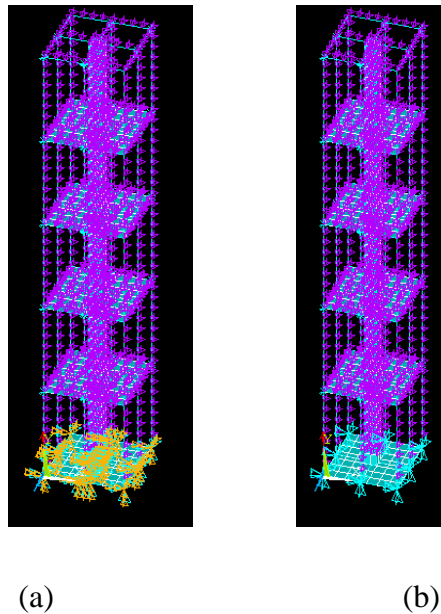


Figure 6-22 Boundary conditions of the finite element model: (a) fully constrained bottom; (b) constrained frame with released core

Result and Discussion

The calibration results of the finite element models are shown in Figure 6-23. It is clear that when the central core of the model is released, the shape of the curve obtained from the finite element transient analysis becomes similar to that from the laboratory test. Moreover, by adjusting the elastic properties of the material, the storey drifts and the force-displacement relationships from the laboratory testing and the finite element analysis match with each other, as shown in Figure 6-23.

6.4.3 Final Configuration of the Model

Based on the analysis of the results, it is clear that the influence of the cable arrangement on the lateral displacement of the model is significant. Also, the bottom fixing conditions of the model is hard to control.

In terms of the laboratory model, to eliminate the influence from the cables, the accelerometers were installed to the model with the cables hanging up to a freestanding steel post. However, the bottom fixing condition of the model can not be improved too much by adding equal angle to the core. Hence, the bottom condition of the model was kept unchanged.

In terms of the finite element model, the bottom of the central core was released so that the laboratory model could be represented.

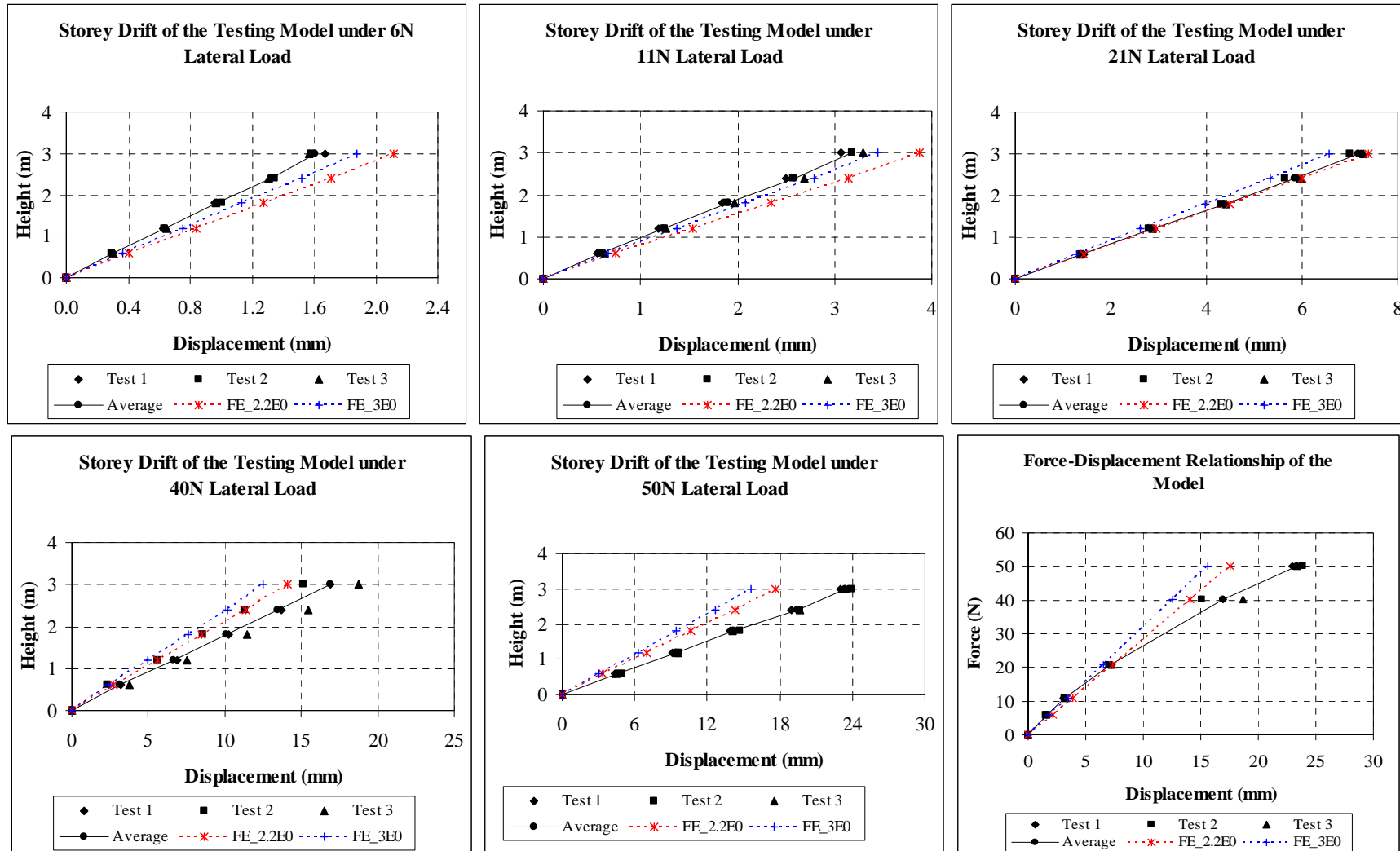


Figure 6-23 Calibration of the finite element model (FE-Finite Element)

6.5 Description of the Laboratory Testing

Through the calibration conducted in previous sections, the final model adopted for the laboratory testing has 5 levels with a base to height ratio of 1:6 (Figure 6-24). The structural frame of the model includes rigidly connected beams and columns (Figure 6-24), with a central core sitting in the middle.

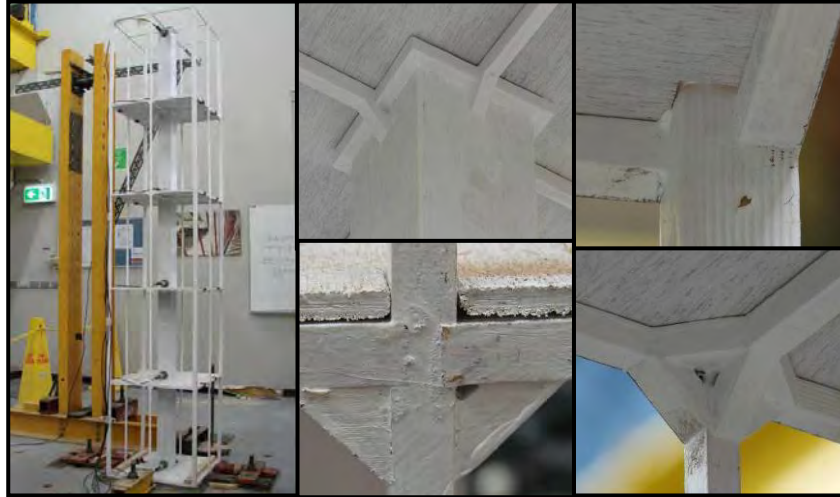


Figure 6-24 The testing model and the beam-column connection details

The measuring system is shown in Figure 6-25. Twelve Dytran 3192A accelerometers were installed to the model, six for the measuring purpose and the other six for the balancing purpose. At each level, one accelerometer was attached to the front left corner of the model as the measuring accelerometer with another one attached to the diagonal corner at the back of the model for the balancing purpose. All the cables were hung up to a freestanding steel post next to the model in order to eliminate the influence caused by cables. Apart from the accelerometers, other devices involved in the laboratory testing include: two signal conditioners, two power suppliers, cables for data processing, a data acquisition card, a computer and data acquisition software (LabView 6.1).



Figure 6-25 Measuring system, Dytran 3192A accelerometer

The loading system includes a crown block set and a string, a freestanding steel post as the support of the crown block set, some weights and the load applying-releasing system (Figure 6-26).



Figure 6-26 Loading system, crown block and the weights

Being conducted in the laboratory, the plug tests include three stages:

- Stage 1: structural skeleton without non-structural panels. At this stage, the structural skeleton was tested under different loading grades: 6N, 11N, 21N, 40N, 50N. There was one measuring accelerometer attached to the left-front corner with a balancing accelerometer at the right-rear corner at each level;
- Stage 2: structural skeleton with non-structural panels attached via double roller connections. At this stage, the infill panels were assembled to the structural skeleton by using double roller connections (Figure 6-27). The model was then tested under the same loading grades and measurement conditions as in stage 1;



Figure 6-27 Details of the double roller connection of infill panels to the frame

- Stage 3: structural skeleton with non-structural panels tightly fixed to the frame. Similar to the second stage, the infill panels at this stage were fixed to the frame of the model. The model with fixed infill panels was tested under the same loading and measuring conditions.

At each stage, the test for each loading grade was repeated three times, to assure the reliability of the results. The structural motion was captured by the measuring accelerometers and then transmitted and recorded by the signal conditioners and the computer. Results obtained from the tests were analysed using Fast Fourier Transform. The raw acceleration-time history was converted to the velocity-time history and displacement-time history. The required information was thus obtained for this study.

The key structural features investigated in this study are the storey drift and the force-displacement relationship of the structure.

6.5.1 Assumptions and Limitations of the Laboratory Testing

In the laboratory testing, the following assumptions were made.

- Assume that the beam-column joints are rigid. Perfectly rigid connections between beams and columns are hardly achievable in the practice, especially in the timber frame structure. However, since the focus of this study is to evaluate the effects of non-structural components on the overall structural performance, to reduce the complexity of the problem, the connections can be considered as rigid;

- Assume that materials behave within the elastic range. Since the applied loads were all within a small scale, and given the repeatability of the test, it is concluded that the materials should work within their elastic range during the experiment;
- Assume that the devices in the loading system have frictionless contact with each other. In this study, frictions between the crown block, the string, the weights and the plug are considered as zero. For example, during the tests, the loading action was considered to be quick enough so that it could be treated as transient action.
- Assume that the unavoidable minor errors caused by manual operation of the measuring system are ignorable.

Limited by the design and the measurement system, some of the structural features are not able to capture/measure, for instance, the stress distribution within different components and the natural frequency of the structure. However, detailed analyses of these features are provided in the following chapter to discuss the theoretical and finite element analyses of tall building structures.

6.6 Results and Discussion

After the completion of the tests at the three stages described in the above section, the preliminary results and findings were obtained, as listed in the following sections.

6.6.1 Testing Results

Stage 1: Skeletal frame without panels

At this stage, the model frame was tested under 6N, 11N, 21N, 40N and 50N lateral loads respectively. Each test was repeated three times to ensure the accuracy and the reliability of the results. A Dytran 3192A accelerometer was attached to the left-front corner of the model at each level to measure the acceleration of the model under different loading conditions, whilst another 3192A accelerometer was set up at the right-rear corner for the balancing purpose only. Cables of the measuring accelerometers were all hung up to a freestanding steel post orthogonal to the loading direction.

Results of the storey drift of the structural skeleton without the infill panels under different loading grades are plotted in Figure 6-28. From the figures, it is identified that

similar readings can be obtained from the three repeated test under every loading grade. This validates the reliability of testing results. Moreover, the repeatability of the tests validates the integrity of the model. It assures that no vital stress dissipation or element damage happen during the test. It is observed that when under lateral loads, linearity appears in the storey drift of the model. This is because of the bottom fixing conditions discussed in previous sections. When the lateral load increases from $6N$ to $50N$, the top displacement of the model increases accordingly, from $1.6mm$ to $24mm$.

The force-displacement relationship of the skeleton model is plotted in Figure 6-28. It is clear that when the lateral load reaches to more than $40N$, non-linearity appears in the force-displacement relationship. The following two reasons are concluded after detailed investigation:

- Imperfect joint connections and possible stress dissipation in some of the joints resulted from the high lateral loads. As discussed in previous sections, it is understandable that since beams and columns are connected by the liquid nails and triangular enhancements, the connections are not 100% rigid. Moreover, the glue between the elements might be crushed when it is under heavy loads;
- Possible stress dissipation in some of the elements under high lateral loads. It is also possible that when the external force is big enough, some micro-cracks/flaws start to develop in the elements. Even though these defections may not be vital, slight influence on the overall structural behaviour can also be reflected.

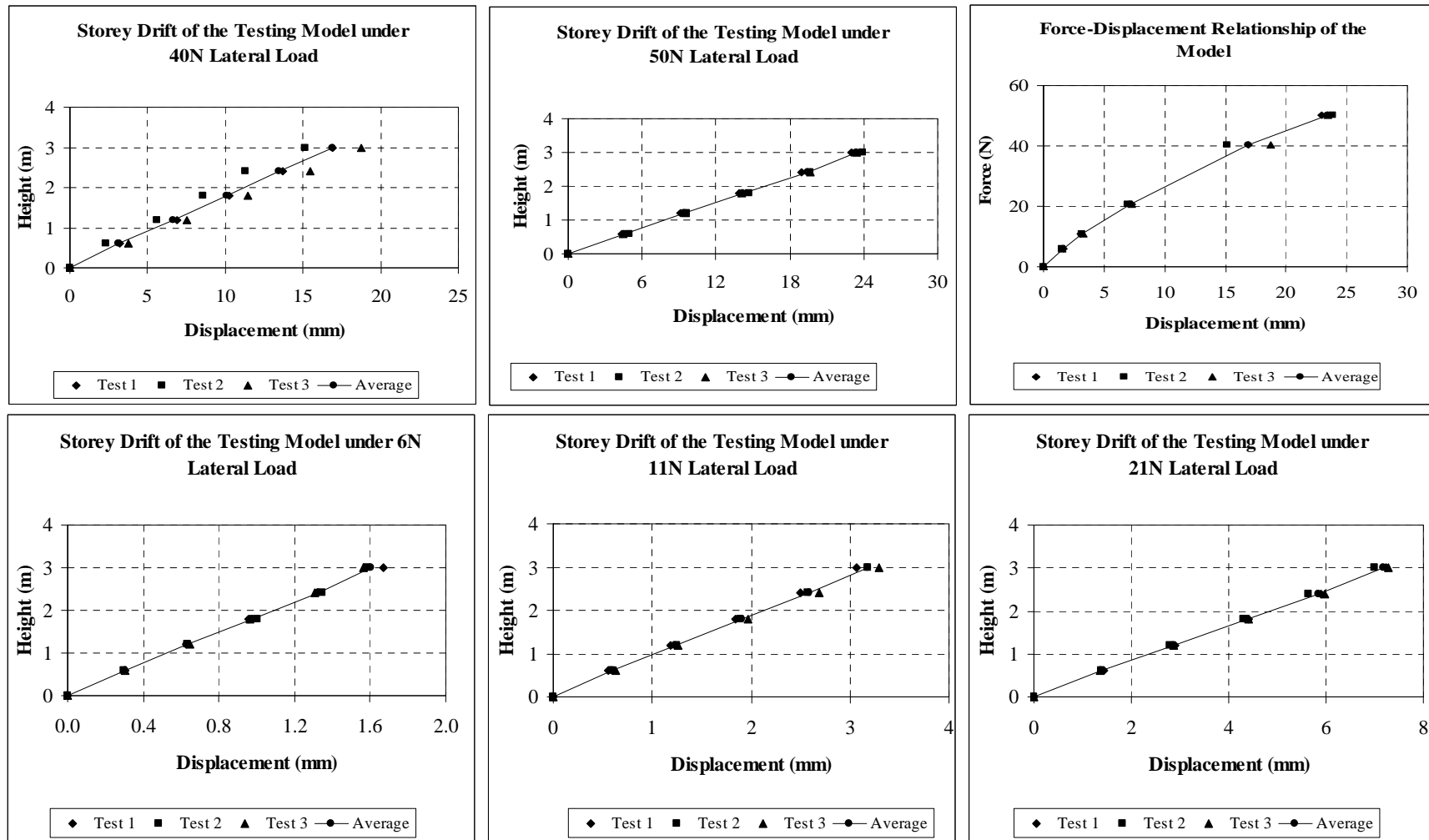


Figure 6-28 The storey drift obtained from the laboratory plug tests on the model without non-structural panels (FE-Finite Element)

Stage 2: Skeletal frame with infill panels attached by using double roller connections

At this stage, infill panels were attached to the skeletal frame along the loading direction by double roller connections. The model was also tested under the loading grades of 6N, 11N, 21N, 40N and 50N. Similar to the stage 1, each test was repeated three times for the accuracy and reliability of the results. The measuring system was set up exactly in the same way in the testing stage 1.

The testing results at this stage are shown in Figure 6-29. Even though the consistency of the results within each loading grade is not as high as that obtained from the stage 1, the discrepancy among the testing results in each group is still within an acceptable range (from 3% to 24%). Moreover, by observing the results carefully, it is noted that the storey drift is also linear. It not only demonstrates the consistency between different stages but also illustrates that the fundamental behaviour of the structure will not change by including non-structural panels to the structural skeleton. Regarding the lateral deflection of the model with panels connected to it by double roller connections, the minimum 0.8mm to the maximum 4.2mm top deflection of the structure can be achieved when the lateral load increases from 6N to 50N. Different to stage 1, from the force-displacement relationship of the model at stage 2, no evidence of minor damages of elements or stress dissipation in connections is identified, even under high lateral loads. This indicates that the non-structural panels can enhance the overall rigidity of the model, and thus can introduce extra stiffness to the structural skeleton.

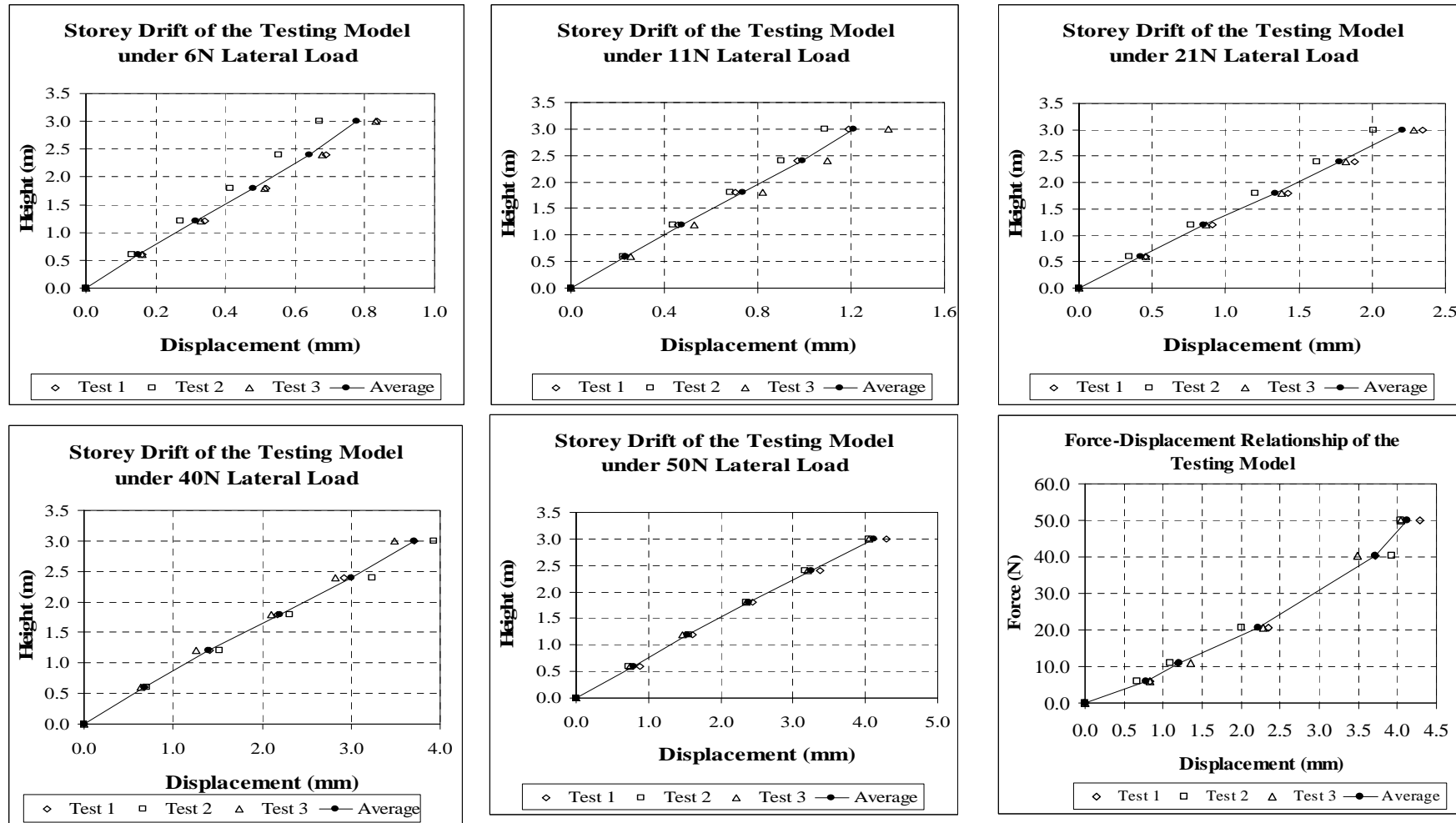


Figure 6-29 The storey drift obtained from the laboratory plug tests on the model with non-structural panels connected to the structure by double roller connections (FE-Finite Element)

Stage 3: Skeletal frame with infill panels tightly fixed to the frame

The infill panels were tightly fixed to the frame at this stage. They were installed along the loading direction. The model was tested under lateral loads of 6N, 11N, 21N, 40N and 50N respectively. Similar to the previous two stages, the test was repeated 3 times under each loading grade. The same measurement system was also adopted.

Figure 6-30 shows the results obtained from the tests at stage 3.

It is observed that under the same loading condition, the frame with infill panels tightly fixed to it has much less storey drift. From the figures, the model drifts from 0.08mm to 0.35mm when the load increases from 6N to 50N.

The force-displacement relationships obtained from the testing results do not show high linearity as in previous stages. The most likely reasons for this phenomenon are:

- (1) With the infill panels tightly fixed to the frame, there are no special connections made between the frame and panels. Under this circumstance, when the load increases, interactions between the structural frame and panels become more distinctive. This makes the panels greatly involved in the structural performance;
- (2) Because of the “infill” characteristic (no special connections to the structure), the behaviour or the interaction between the panels and the frame is, to some extent, random and hard to be predicted.

The combination of these two reasons can lead to the happening of the slightly non-linear force-displacement relationship.

In regard to the repeatability of the tests, each test was repeated three times. From the results shown in Figure 6-30, even though the discrepancies among the results are more obvious than those at previous stages, they are still within an acceptable range (under 20%).

It can tell from the results the overall structure becomes very stiff that because of the inclusion of infill panels. Moreover, resulted from the interaction between the structural frame and the infill panels, as well as the unpredictable racking of the panels owing to the lack of connectivity to the frame, the testing results fluctuate comparing with other tests conducted at previous stages.

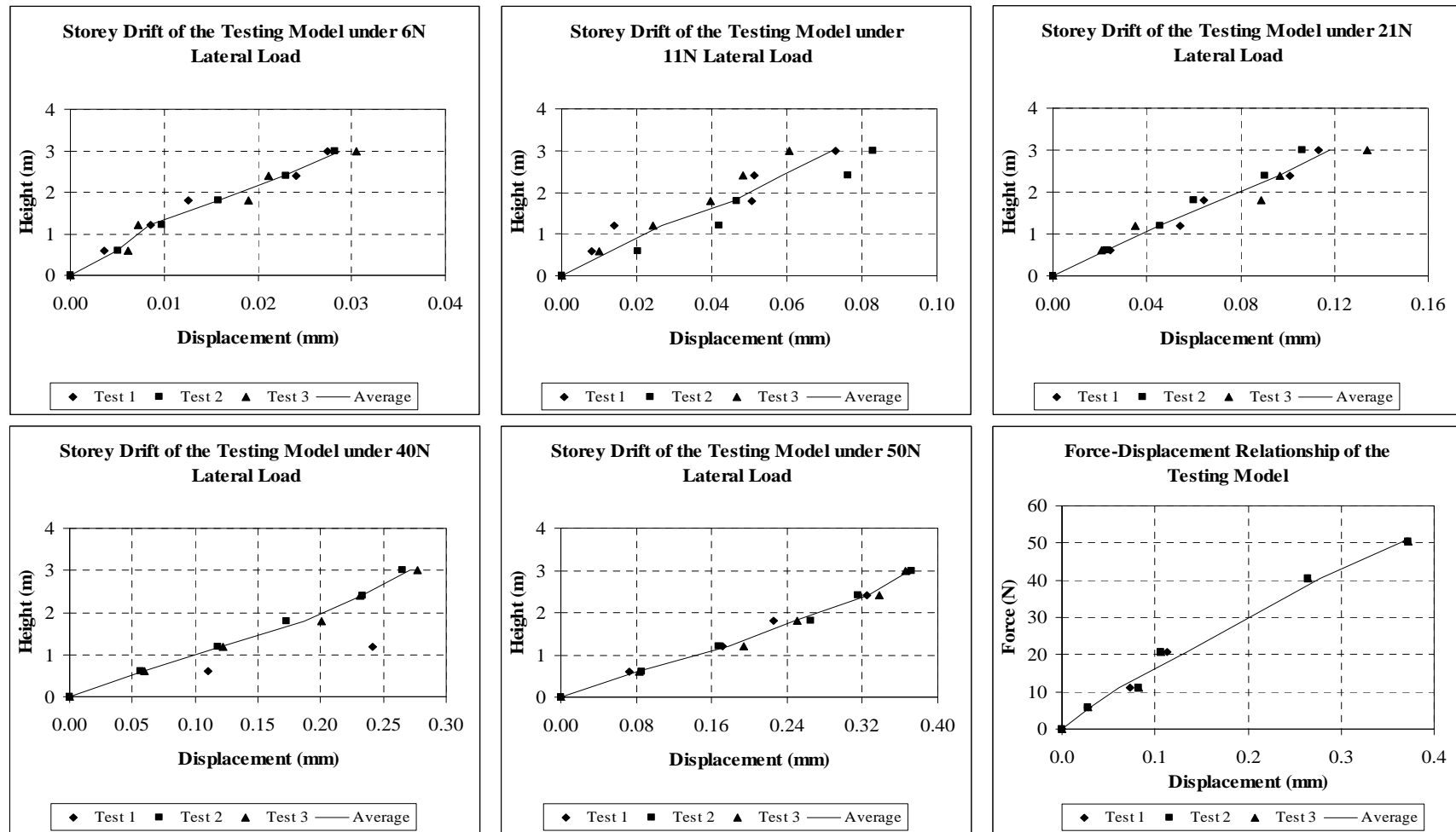


Figure 6-30 The storey drift obtained from the laboratory plug tests on the model with non-structural panels tightly set in the structure frame (FE-Finite Element)

6.6.2 Comparison and Discussion on the Laboratory Testing Findings

By comparing the testing results of all the three stages, it can be concluded as shown in Figure 6-31 that the inclusion of infill panels increases the structure stiffness significantly.

In Figure 6-31, the force-displacement relationships of the structure with and without non-structural panels are compared. It is obvious that when the non-structural panels are connected to the structural frame by using double roller connections, the top deflection of the overall structure under 20N lateral load is reduced from 7.3mm (structure without non-structural panels) to 2.2mm (structure with double-roller-connected non-structural panels) whilst the reduction is from 23.5mm to 4.2mm when under 50N lateral load. If those non-structural panels are tightly fixed to the structural frame, a further decrease of the top deflection of the structure can be achieved. Under 20N and 50N lateral loads, the top deflection of the model with non-structural panels tightly fixed to the structural frame decreases to 0.12mm and 0.37mm respectively. If translating into the amount of contribution in percentage, as shown in Figure 6-32, it can tell that constant contribution is made to the structural stiffness by including non-structural panels to the structure. When the panels are attached to the structural frame by using double roller connections as introduced in this study, more than 40% of stiffness increase (from 47% to 82%) can be obtained. Moreover, when the panels are tightly fixed to the frame, their stiffness contribution will further reach to a constant 95%. It not only demonstrates that the stiffness of the structure can be enhanced by including non-structural components to it, but also proves that the stiffness contribution of the non-structural components to the structural performance is greatly influenced by the connection properties.

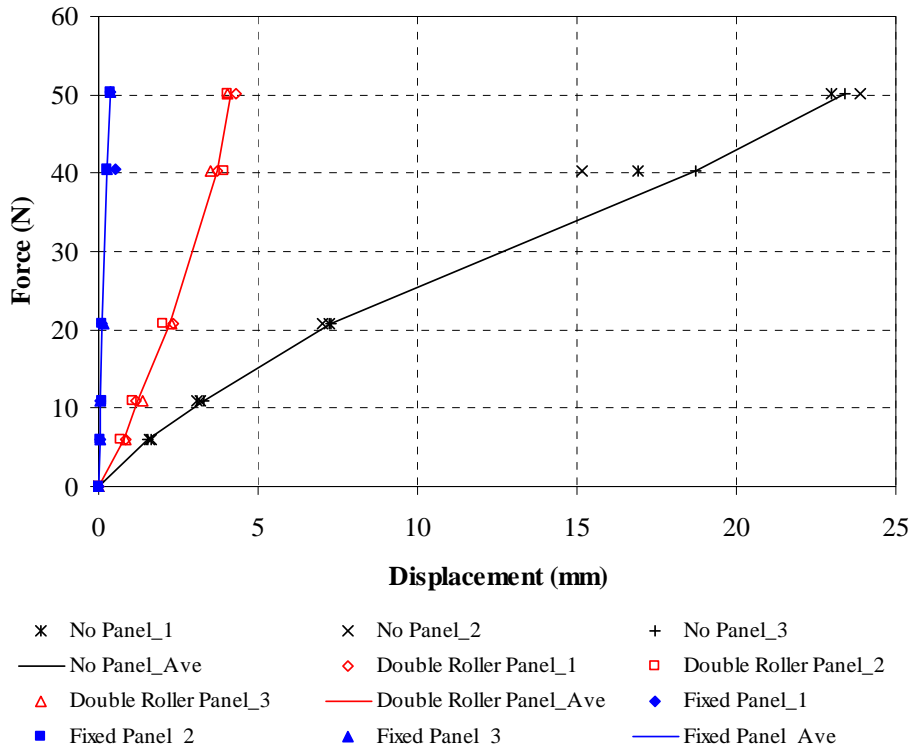


Figure 6-31 Comparison of the force-displacement relationships obtained from the laboratory plug tests on the model with and without non-structural panels (FE-Finite Element)

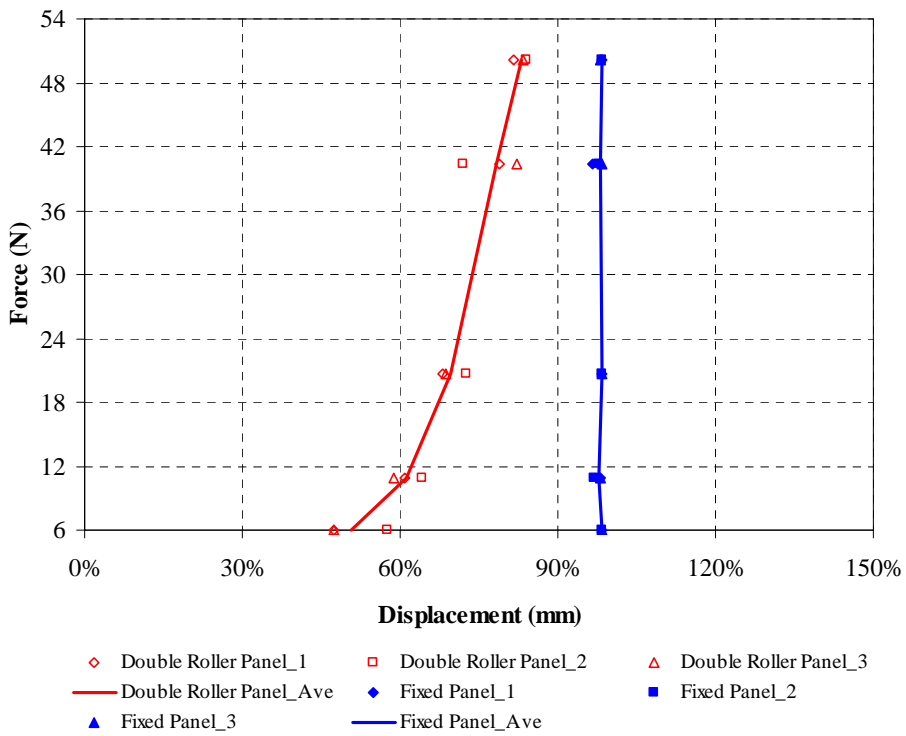


Figure 6-32 Evaluation of contributions from the non-structural panels to the global performance of the structure by analysing the laboratory testing results (FE-Finite Element)

6.6.3 Static Analysis Conducted on Finite Element Models

Static analysis was also conducted by using finite element models, to ensure that the decrease of the storey drift measured in the above tests is not because of the introducing of extra mass to the overall structural system.

The finite element model calibrated in previous sections was adopted (Figure 6-33(a)). A lateral load was applied at the top centre of the structural core (Figure 6-33(b)). The boundary conditions of the model are the same as defined in the transient analyses, with released central core (to represent the imperfect bottom fixing condition of the laboratory model) and rigid beam to column connections. Similar to the laboratory tests, there are three stages in the finite element static analysis: (a) structural skeleton without non-structural panels (Figure 6-33(b)); (b) structural skeleton with non-structural panels connected to it by double roller connections (Figure 6-33(c)); (d) structural skeleton with non-structural panels tightly fixed to the structural skeleton (Figure 6-33(d)).

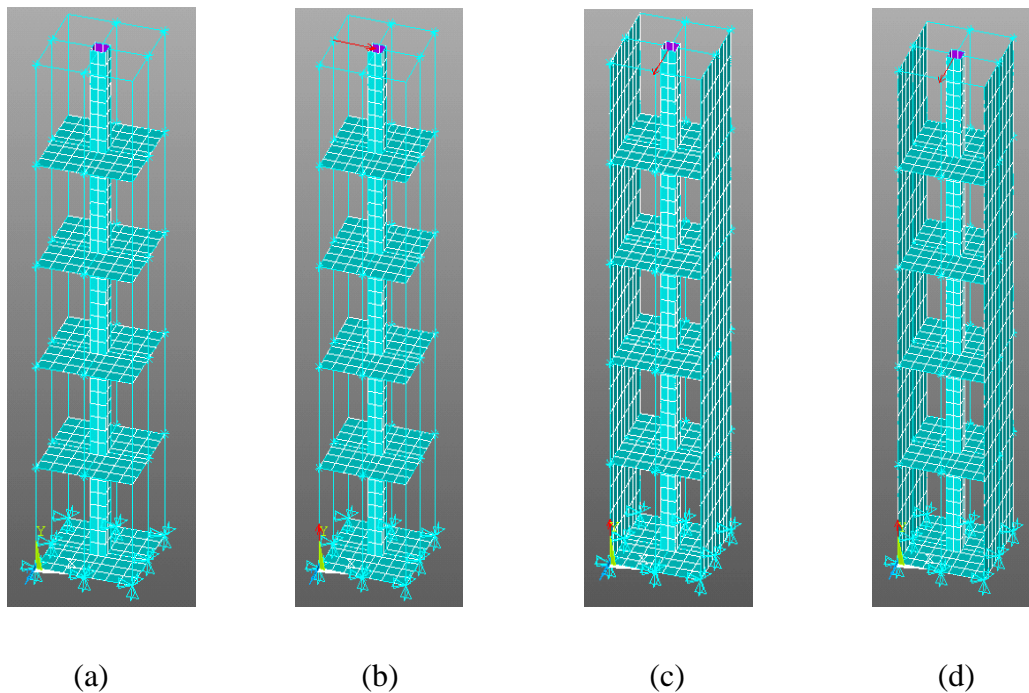


Figure 6-33 Finite element models for the static analysis at different stages: (a) the calibrated structural model; (b) structural model without non-structural panels; (c) structural model with non-structural panels connected to it by double roller connections; (d) structural models with non-structural panels tightly fixed to the frame

Figure 6-34 plots the comparison of the force-displacement relationships of the structural model under different configurations. It can be observed from the results that

when under the static analyses, the stiffness increase caused by inclusion of non-structural panels has exactly the same trend as that in the laboratory testes. When subjected to the lateral loads from $6N$ to $50N$ applied at the top of the structure, the top deflection of the model without the non-structural panels is $15.71mm$ whilst the models having non-structural panels with pinned and fixed connections deflect to $8.78mm$ and $0.33mm$ respectively. Figure 6-35 attempts to quantify the amount of contribution of the non-structural panels connected to the structure in different ways. It shows clearly that a constant 44% stiffness increase is achieved by pin-connecting the non-structural panels to the structure. Likewise, when the panels are fixed to the structural frame, the stiffness contribution can reach to 99%.

The findings in the static finite element analysis are consistent with that in both the finite element transient analysis and the laboratory plug test. This again demonstrates that the non-structural components can add extra stiffness to the structure during its service life.

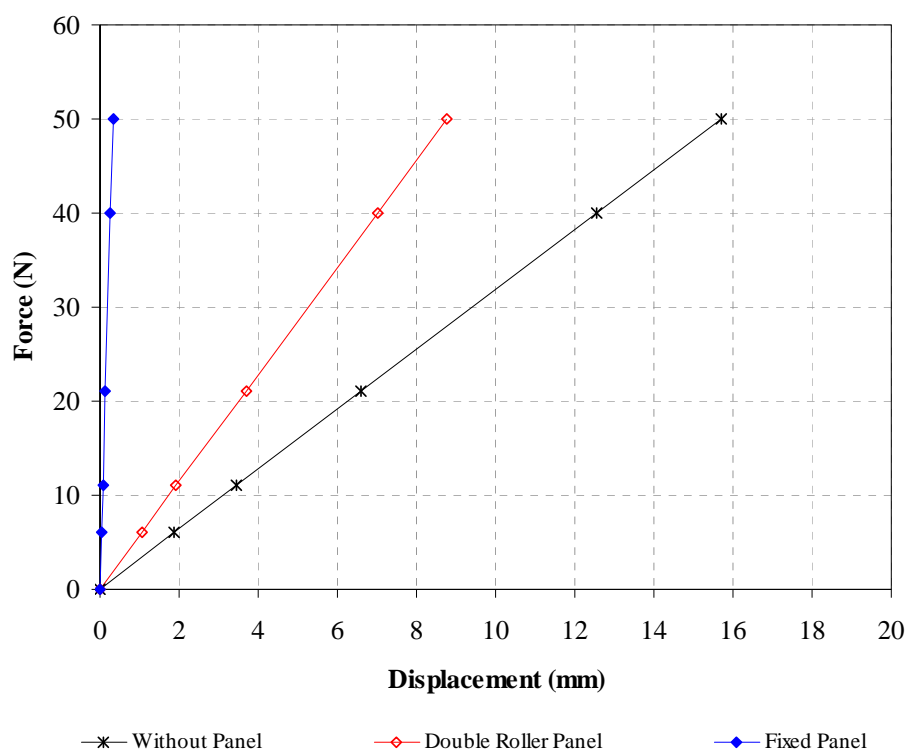


Figure 6-34 Comparison of the force-displacement relationships obtained from the finite element static analysis on the model with and without non-structural panels (FE-Finite Element)

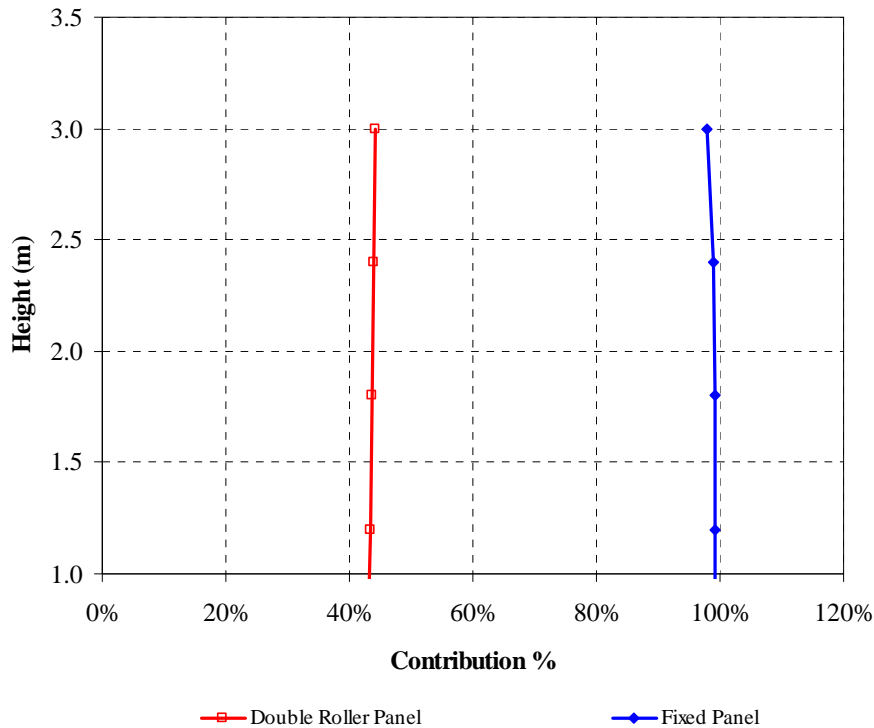


Figure 6-35 Evaluation of contributions from the non-structural panels to the global performance of the structure by analysing the finite element static analysis results (FE-Finite Element)

6.7 Conclusions

It is widely adopted in the theoretical analysis of tall building structures that the non-structural components are articulated from the skeletal frame whilst can hardly be implemented in the practice. Thus, contributions of non-structural components to the overall structural performance of tall buildings may have been overlooked. Besides, the amount of loads attracted by the non-structural components may also be underestimated. These will directly lead to the pre-failure of non-structural components and the uneconomical design of tall buildings as an assemblage of strong primary structures and delicate/brittle non-structural components.

The laboratory testing discussed in this study demonstrates the differences of building behaviour caused by including non-structural components.

Based on the results obtained from both the laboratory tests and the finite element analyses, some key conclusions are drawn as follows:

- The stiffness of a tall building structure can be significantly increased by including non-structural components in the analysis. Based on the laboratory testing in this study, a maximum 98% stiffness increment can be realised by setting non-structural panels tightly to the structural frame;
- The amount of the stiffness increase is also influenced by the connection properties between the non-structural components and the primary structure. In this study, when attaching non-structural panels to the structure by using double roller connections, the reduction of the storey drift of the overall structure is only about half of that when under fixed connections;
- The finite element model and the modelling technique developed according to the testing results in this study have high confidence of being used in the future analysis. In the analyses, the finite element model was calibrated by the testing results obtained from the laboratory model with and without structural panels. To validate the reliability of the model and the conclusions drawn from the transient analyses, a series of static analyses were also conducted. The results appear to be highly consistent.

It is noted from both the laboratory testing and the finite element analyses that a significant increase of structure stiffness can be obtained by introducing non-structural components to the structural analysis. As introduced in Chapter 2, non-structural components are normally excluded from the structural analysis and detached to the primary structure in the design and construction practice hence their effects on the overall behaviour of the structure are not taken into account in the current practice. Under this circumstance, this study may reveal potential benefits to the building industry by considering non-structural components in the structural analysis.

However, conclusions drawn so far in this study are based on the tests and analysis conducted to the laboratory model. Some of the structural performance, such as the natural frequency and the stress distribution within different non-structural components, can not be fully evaluated through the test. Given these findings, a comprehensive analysis of the full-scaled model has been conducted, in order that the findings from this study may be developed into a process for an improved design approach. Detailed theoretical and finite element analyses follow in Chapter 7.

CHAPTER 7

THE ANALYSIS OF TALL BUILDINGS

7.1 Introduction

It has been established that current Australian high-rise building design practice is to assume that the structural skeleton of a building provides resistance to any lateral forces that might occur. The overall design of high-rise buildings is usually dominated by serviceability limit state considerations rather than the ultimate limit state factors.

The diversity of structural and component properties, as well as connections result in a level of complexity. That requires numerous assumptions in the theoretical analysis of multi-storey buildings. For instance, in a rigid frame structure, structural skeleton is considered the lateral load resisting system of the building. It directly determines the overall building performance. However, in a core-frame structure, the concrete service core works together with the structural frame to resist the lateral loads, and consequently, leads to different building performance comparing with the rigid frame structure.

It has been identified that various structural forms and materials used in the construction of high-rise buildings result in different structural responses. It has also been established that current theoretical design methods are lacking in inputs to the design of non-structural components. The form and materials utilised in the design and also the interaction between structural components and non-structural components for that buildings are widely recognised as a complex assemblage of both structural skeleton and non-structural components (Su et al. 2005). The lateral performance of high-rise buildings is complex because of the conflicting requirements of diverse (structural and non-structural) building systems (Hutchinson et. al. 2006). There is scope to improve the serviceability limit state design requirements over the traditional approach.

The field reconnaissance, case study building analysis and the laboratory tests conducted in this study demonstrate that the structural role of non-structural components, such as façades and infill walls, in resisting the lateral loads is significant and can add extra stiffness to the tall building. Hence, the evaluation of the contribution

from the non-structural components to the overall building performance is necessary. This can provide more detailed information to the design of high-rise buildings.

In this chapter, to provide a clear view and a better understanding of the overall building performance, detailed analyses of different assemblages of structural and non-structural components were carried out both theoretically and by using finite element analysis tools. The influence of non-structural components on the structure performance was evaluated. Meanwhile, the stress distribution within the individual non-structural component was also investigated to demonstrate the damage level of different non-structural components.

Based on the theories validated from the above analyses, a parametric study was developed. The structural system of a case study building with diverse non-structural components was analysed and more sophisticated evaluation of the effects of non-structural components on the overall building performance was achieved. Finite element models were adopted to identify the influence of non-structural components on the lateral stiffness of high-rise buildings, with an emphasis on the serviceability design. Parameters representing non-structural components were changed in turn, to replicate the reasonable variation which can be found in current Australian practice.

There are five stages in the analysis:

- Analysis of the bare frame;
- Analysis of the frame with shear cores;
- Analysis of the core-frame with infill walls;
- Analysis of the core-frame with shear walls;
- Analysis of the core-frame with shear walls and façade system.

At each stage, the storey drift, natural frequency, bending moment and shear force distributions in the columns were investigated.

7.2 Evaluation of Interactions between Structural and Non-structural Components

In this section, detailed analyses of different parts of structures were conducted so that

the contributions of different non-structural components to the overall structural performance could be evaluated. The damage levels of the non-structural components were also analysed so that the guidelines for accounting the actual influence of non-structural components to the structural performance could be proposed. The multi-storey building system involved in this analysis includes the following components: a structural frame, coupled shear walls, infill walls, façade panels, doors and windows.

7.2.1 Identification of the Structural System

7.2.1.1 Assumptions

In order to achieve the maximum reliability of the results via the simplest analysis, some assumptions of the whole theoretical and finite element analysing system were made:

- Structural system: the structure is assumed to be symmetrical;
- Materials: any plastic/non-elastic behaviour of steel and concrete is excluded. The contribution of any steel reinforcement in the concrete material is not included;
- Loading conditions: the static lateral point loads are applied to the beam-column joints along one face of the structure;

7.2.1.2 Detailed Methodology for the Theoretical and Finite Element Analysis

Comparison of theoretical and finite element analysing results can assure the reliability of the study. In this analysing system, five stages were proposed in order to identify stiffness contributions from different non-structural elements to the overall structure performance. They are:

- The analysis of the bare frame structure;
- The analysis of the frame with infill walls;
- The analysis of the coupled shear wall;
- The analysis of coupled shear wall with windows;

- The analysis of the shear wall frame with façades.

Within each stage, parallel analyses based on both structural theory and finite element computer simulations were conducted and then compared. If the results from both parts were comparable or identical, periodical conclusions could be drawn and the holistic process could move to the next stage. Otherwise, the whole process of this stage should be revised and re-conducted. Details of the analysing process are shown in Figure 7-1.

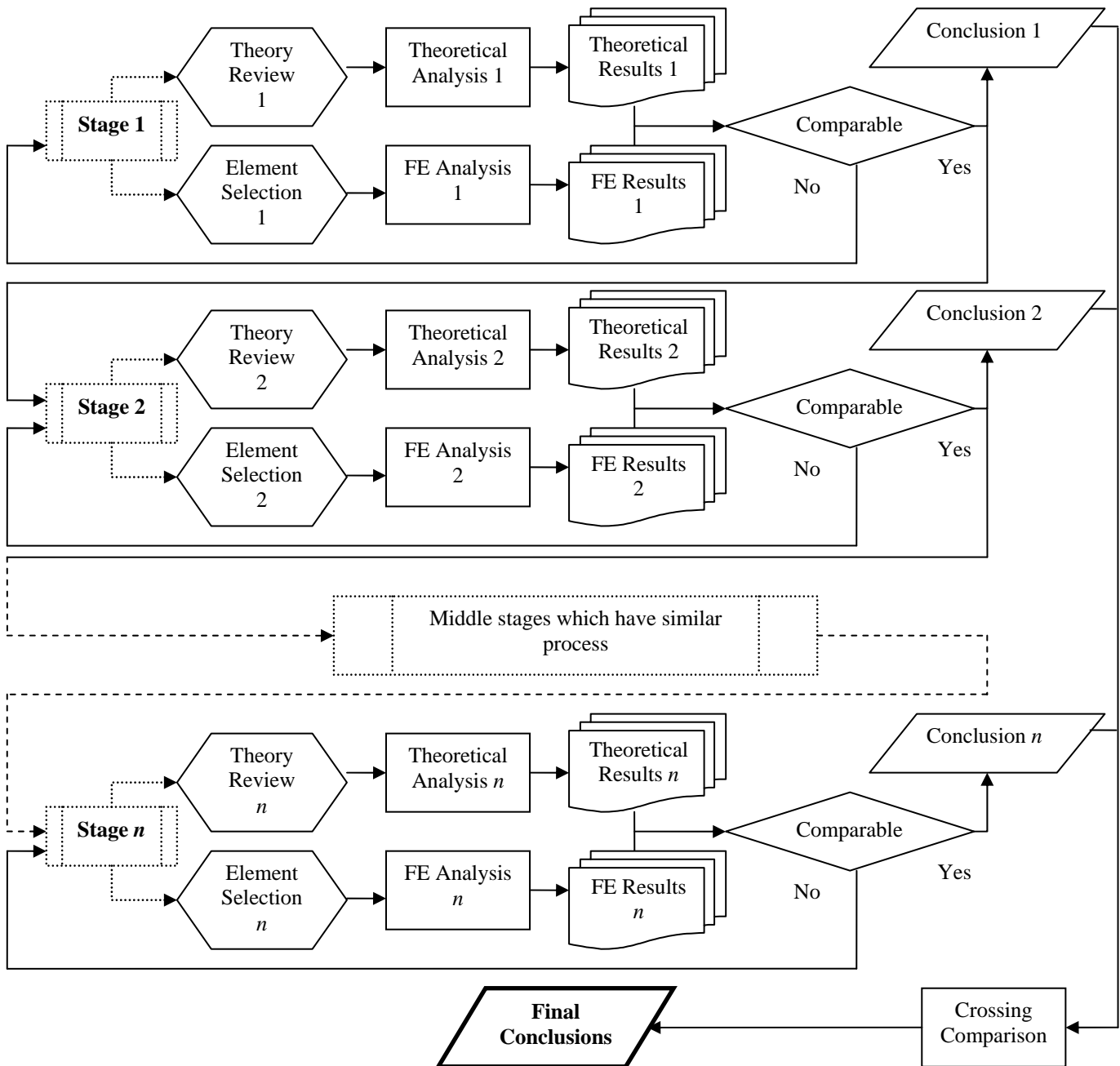


Figure 7-1 Flow chart of the analysing process

7.2.1.3 Structural Elements and Properties

This study is based on a typical case-study building that has a $30m \times 30m$ floor plan with main column supports at every $10m$ (Figure 7-2(a)). The total height of the building is $90m$, with $3m$ storey heights over 30 storeys. To simplify the analysis, a uniform cross section steel beam 360UB56.7 was assigned to the beams and a steel 310UC118 column to the columns. The thickness of the coupled shear wall is $0.4m$ (Figure 7-2(c)). The $0.1m$ precast concrete panel was introduced to the analysis as infill walls (shown in Figure 7-2(b)). The gap between the frame and the infill wall is very small and no special bonds or connections were made between the infill walls and the structural frame, only contact elements were defined in the finite element model to assure the contact between the frame and infill walls. The façade system is composed of aluminium façade frames and glass panels (Figure 7-2(d)). The cross sectional area of an aluminium façade frame is $0.1m$ by $0.03m$. The thickness of the glass panels is $0.02m$. The windows are considered to have $0.01m$ thick glass panels with $0.1m$ by $0.05m$ aluminium frames. Details of the geometric and material properties of these structural and non-structural components are listed in Table 7-1. ANSYS models were developed accordingly.

When doing an analysis for assemblies of components, different parts of the structural frame were chosen in order to identify the contributions from various non-structural components. For example, in the analysis of the infill wall frame performance, a single-span multi-level and multi-bay 3-D frame (Figure 7-2(b)) was chosen to identify the stiffness contributions from the infill wall (Figure 7-2(b)). When evaluating the contributions from doors and windows, a coupled shear wall was analysed separately. Details of the configurations for every stage are discussed in the following sections.

7.2.2 Modelling Details

7.2.2.1 Bare Frame Analysis

Since any effect of the foundation is beyond the scope of this study, all six degrees of freedom (DOF) of the structure base were fully constrained in order to match with the widely adopted “cantilever” theory in tall building analysis. Three-node 3-D beam elements (BEAM4) were used to represent both columns and beams. The connections between beams and columns were considered as rigid. Details of the ANSYS elements are provided in Table 7-1.

7.2.2.2 Frame with Infill Walls

Precast concrete infill walls were involved in the analysis. To model the connection between infill walls and the frame, a very small gap was defined as shown in Figure 7-2(b). However, in real practice, the infill wall is normally built as a built-in wall. Four-node 3-D shell elements (SHELL63) were used to represent the infill walls, and the point-to-point contact elements (CONTAC52) were created to simulate the connectivity across gaps. Details of elements are listed in Table 7-1.

7.2.2.3 Shear Wall Frame with Façade Panels

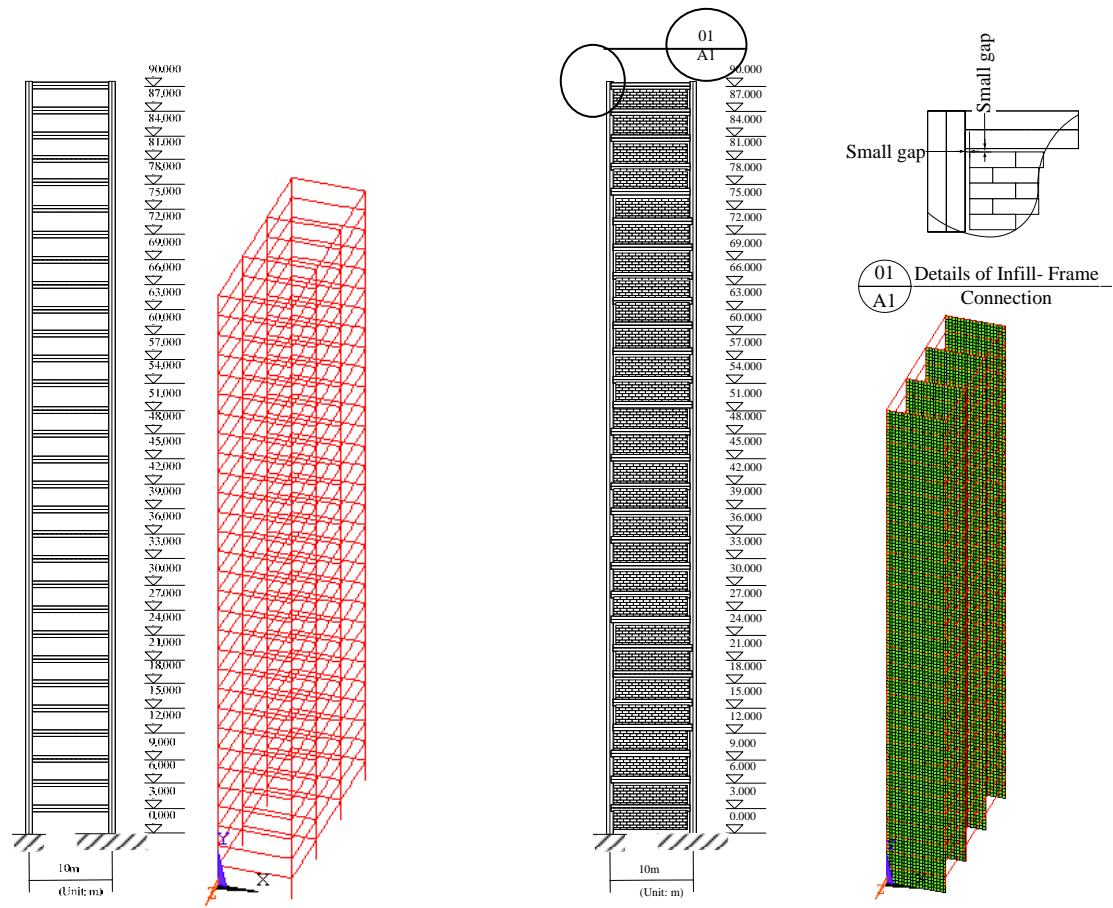
In the finite element analyses, the same shell element (SHELL63) was adopted for the glass panels of the façade system. The 3-D beam elements (BEAM4) were used to model the aluminium façade frame. The façade system was considered as a built-in floor-to-ceiling façade as shown in Figure 7-2(d). Table 7-1 presents the details of the elements and materials allocated in the analysis.

7.2.2.4 Coupled Shear Wall Analysis

Concrete coupled shear walls with dimensions of $10m \times 90m \times 0.4m$ were modelled by shell elements (SHELL63) (Figure 7-2(c) and Table 7-1). Universal openings ($2m \times 2m \times 0.4m$) were located at the middle of each span at each level of the wall (Figure 7-2(c)).

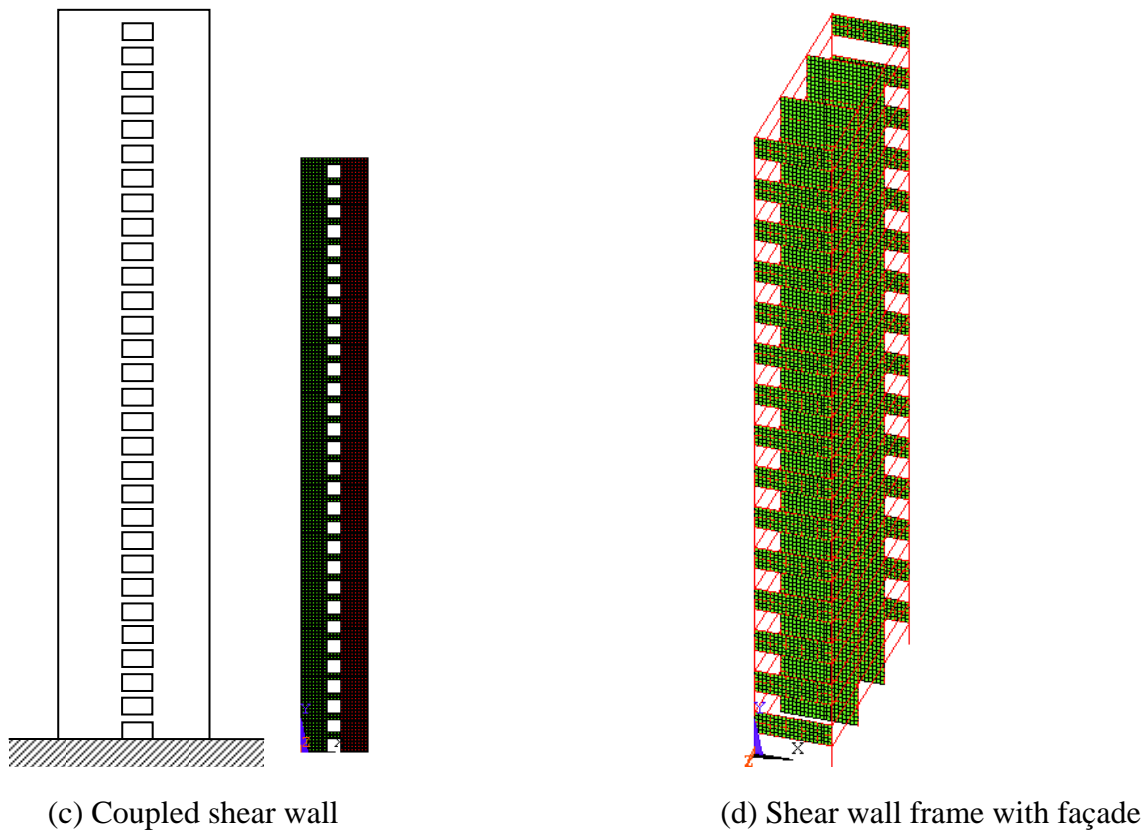
7.2.2.5 Coupled Shear Wall with Windows

Windows were included in the second step analysis of the coupled shear wall in order that the stiffness contribution of the windows could be evaluated. The glass panels were represented by shell elements (SHELL63) and the aluminium window frames were modelled by beam elements (BEAM4) (Table 7-1).



(a). Bare frame

(b) Frame with infill walls


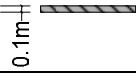
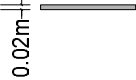

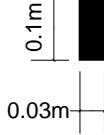



(c) Coupled shear wall

(d) Shear wall frame with façade

Figure 7-2 Configurations of the structure

Table 7-1 Element and material details

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column	360UB56.7	Steel	Linear Elastic $E = 2.0 \times 10^{11} Pa$ $\mu = 0.29$	BEAM4
Beam	310UC118		$density = 7850 kg/m^3$	BEAM4
Shear Wall		Concrete	Linear Elastic $E = 2.5 \times 10^{10} Pa$ $\mu = 0.15$ $density = 2400 kg/m^3$	SHELL63
Infill Wall				PLAN42
Façade Panel		Glass	Viscoelastic $G_0 = 2.74 \times 10^{10} Pa$ $G_b = 6.05 \times 10^{10} Pa$ $1/\beta = 0.53$ $density = 2390 kg/m^3$	SHELL63
Window Panel				
Façade Frame		Aluminium	Linear Orthotropic $E_x = 3.07 \times 10^{11} Pa$ $E_y = 3.58 \times 10^{11} Pa$ $E_z = 3.58 \times 10^{11} Pa$ $\mu_{xy} = \mu_{yz} = \mu_{xz} = 0.2$ $G_{xy} = G_{yz} = G_{xy} = 1.269 \times 10^{11} Pa$	BEAM4
Window Frame				
Connection		--	k	CONTAC52

7.2.3 Results and Discussions

7.2.3.1 Bare Frame Analysis

The deflection at the top of a multi-storey frame under service loads can be considered as the accumulation of the storey drifts up the structure, i.e. the sum of the individual storey drifts caused by column and girder flexure. This is represented by Equations 7-1 to 7-6 below developed by Stafford-Smith and Coull (1991). The nomenclature of the equations is provided at the beginning of this thesis.

$$\delta_g = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L} \right)_i}$$

7-1

$$\delta_c = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h} \right)_i} \quad 7-2$$

$$\delta_{if} = h_i A_0^i \quad 7-3$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \quad 7-4$$

$$\delta_i = \delta_g + \delta_c + \delta_{if} = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L} \right)_i} + \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h} \right)_i} + \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \times h_i \quad 7-5$$

$$\Delta = \sum \delta_i \quad 7-6$$

Figure 7-3 compares the results of the frame from both the theoretical calculations and the finite element analysis. Lateral loads with the sum equivalent to $48kN$ were uniformly distributed to the beam column connections along one face of the structure at every level. The theoretical result and the finite element result show high consistency. When the lateral load reaches about $48kN$, the displacement predicted by the theory is $51.4mm$ comparing with $54.1mm$ from the finite element analysis result. The difference between these two sets of results is 5%, which is satisfactory for the structure under consideration. Similarly, identical results from finite element analyses and theoretical analyses are also achieved in the storey drift analysis. Further more, the force-displacement relationships plotted by both the theoretical and the finite element analyses show high linearity as would be expected, given the assumptions and the linear elastic material properties.

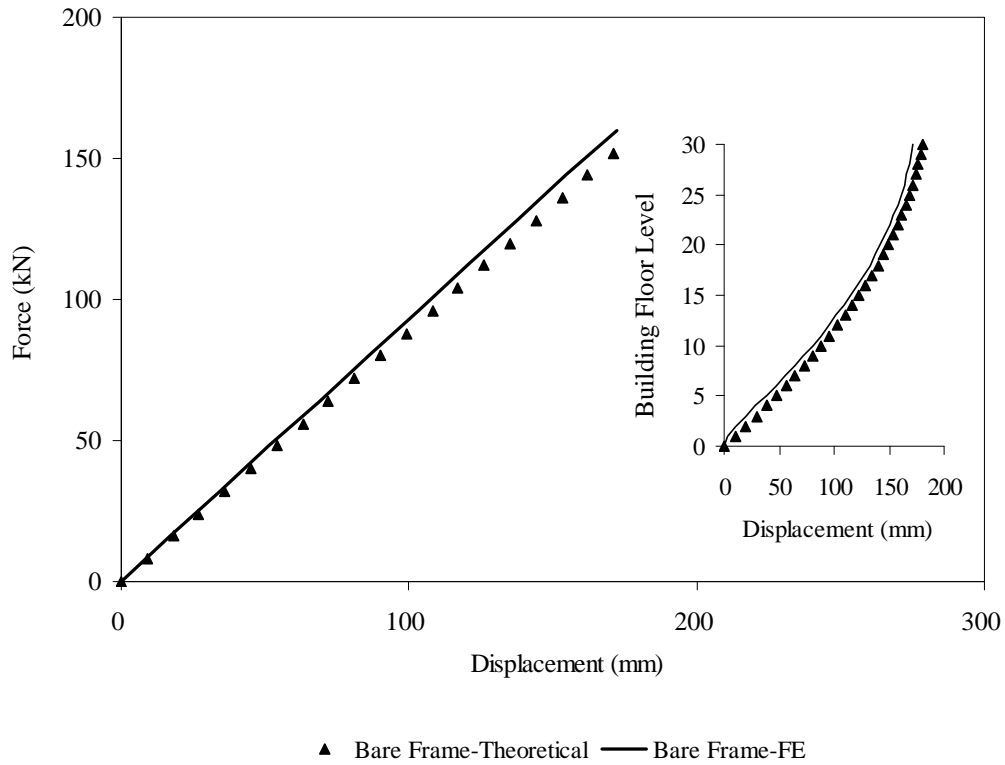


Figure 7-3 Force-displacement relationship and the total storey drift of bare frame analysis

7.2.3.2 Frame with Infill Walls

According to Stafford-Smith and Coull (1991), a frame with infill walls should be analysed as an equivalent bracing system. When under the service load, the top deflection of the braced frame will be the sum of the drift in each storey. The drift in storey i is a combination of deflection caused by the shear deflection of the braced bents at storey i and the flexural column and beam drifts at that storey (Equation 7-7 to 7-11).

$$\delta_s = \frac{Q_i}{2E} \left(\frac{d^3}{L_i^2 A_d} \right) \quad 7-7$$

$$\delta_{if} = h_i \theta_{if} = h_i A_0^i \quad 7-8$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_0 - h_{i-1/2}}{2} \quad 7-9$$

$$\delta_i = \delta_s + \delta_{if} \quad 7-10$$

$$\Delta = \sum \delta_i \quad 7-11$$

As shown in Figure 7-4, a discrepancy of the force-displacement relationships exists between the theoretical analysis and the finite element analysis. This discrepancy indicates that, the finite element model, simulates the extra stiffness caused by the contact between the infill walls and the structural frame only if the contact has happened whilst the theory used in solving the infill wall frame structure considers infill walls as braces and analyses the infill wall frame as a braced frame from the very beginning of the analysis.

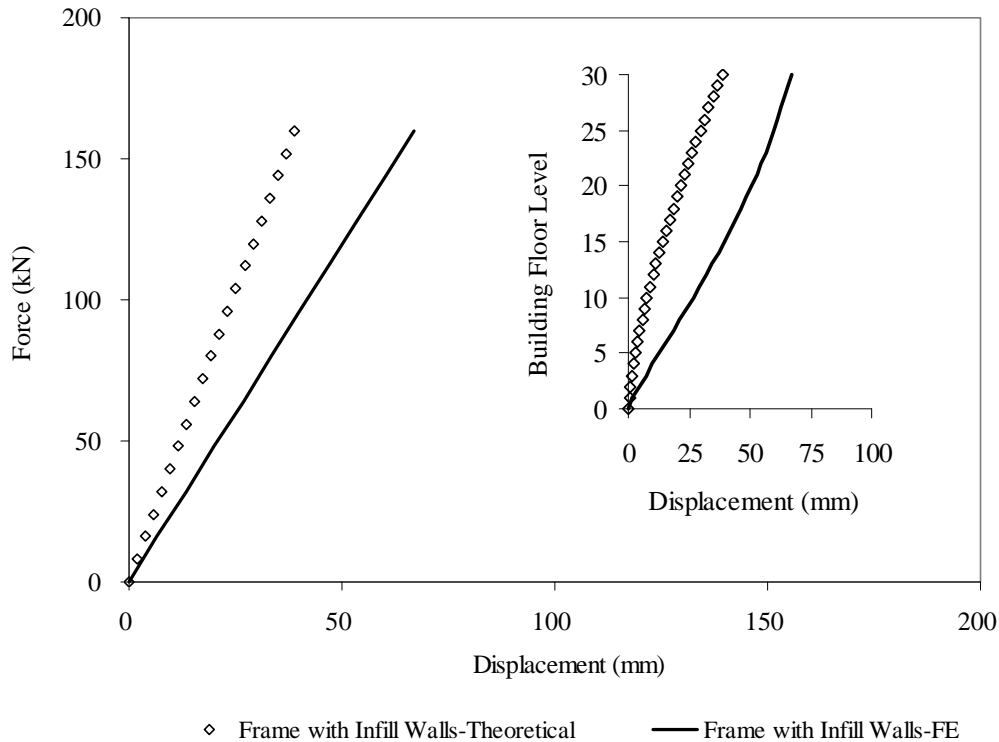


Figure 7-4 Force-displacement relationship and the total storey drift of frame with infill wall analysis

The stiffening effect of infill walls

By comparing the force-displacement relationships and the storey drifts of the bare frame and frame with infill walls as shown in Figure 7-5, it is clear that when under the same loading conditions, more than 60% increase of the structural stiffness can be obtained by including infill walls into the analysis. This leads to a consequent decrease of the top deflection and the storey drifts of the structure. This indicates that significant increase of the structural stiffness can be realised by adding infill walls to the structural frame.

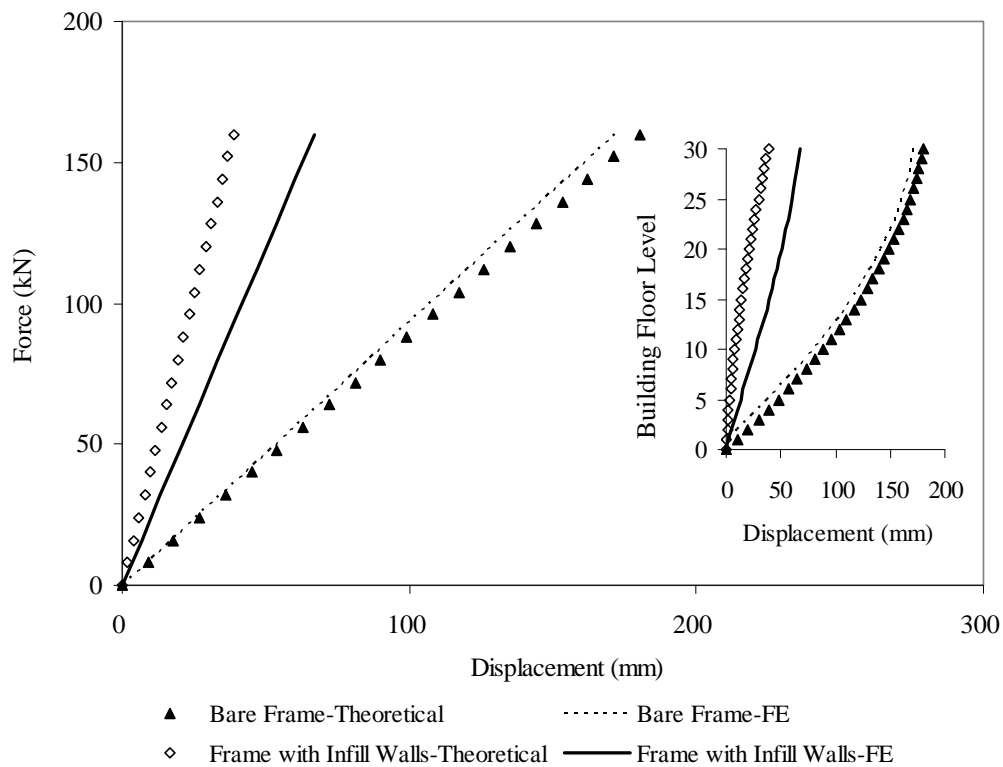
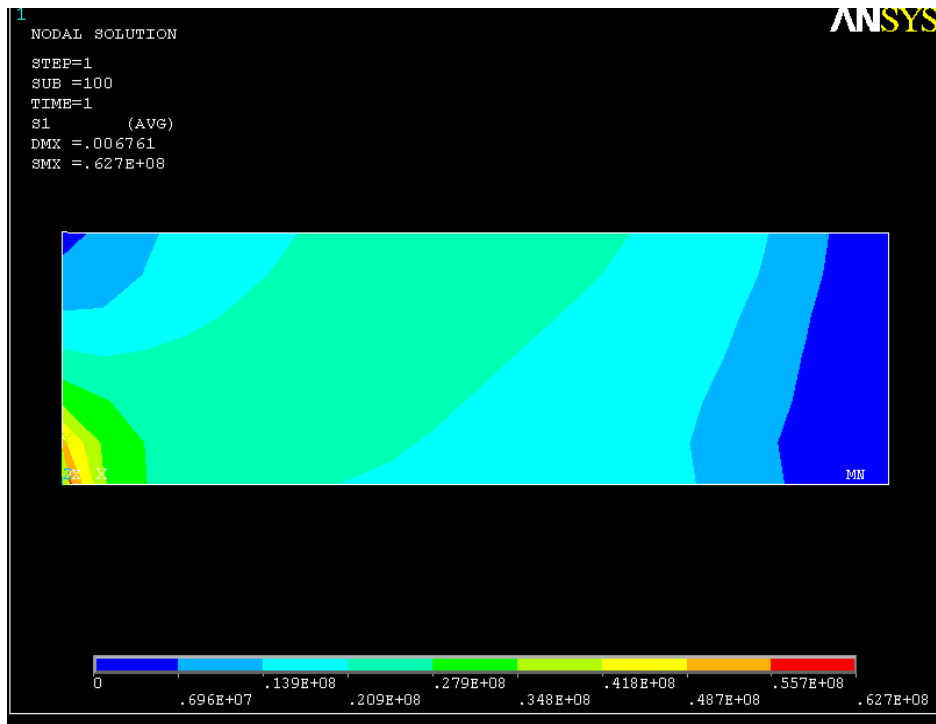


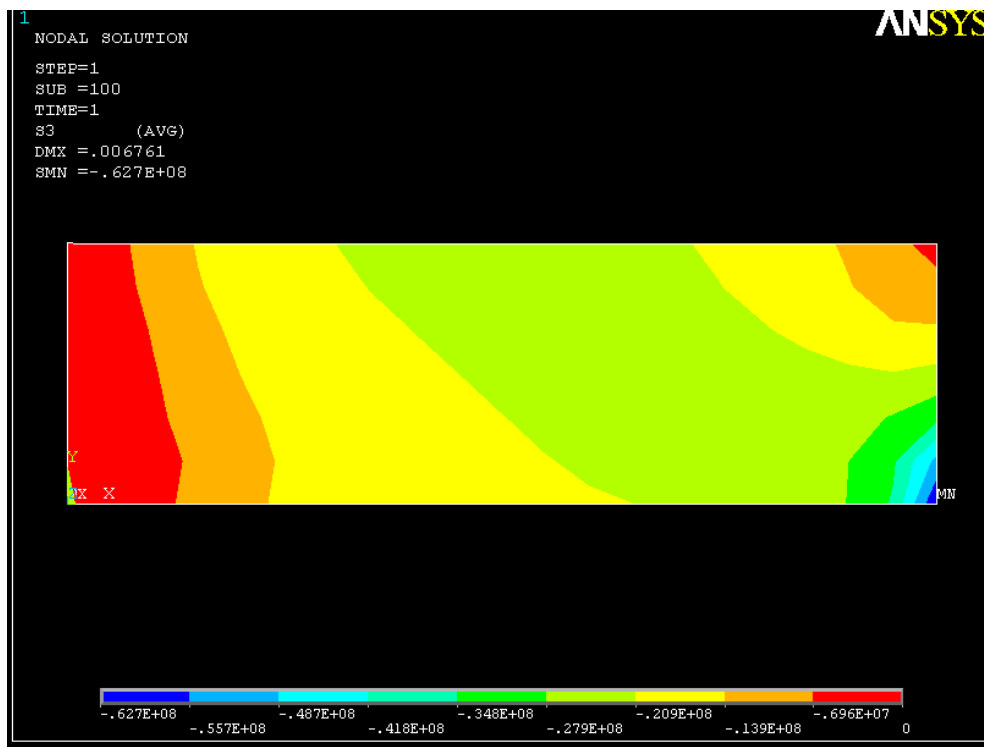
Figure 7-5 Comparison of the force-displacement relationship and the total storey drift of frame with and without infill walls

The stress distribution within the infill walls

At the time considerable increase of the structural stiffness being achieved by including infill walls into the structural analysis, the load redistribution within components caused by the integration of non-structural components also introduces dramatic changes to the stress distribution in the infill walls. In this analysis, the extreme scenario, in which infill walls have direct contact with structural frame at all times when subject to the maximum allowable service level movement, was investigated. Figure 7-6 plots the stress distribution within infill walls when the wall is subject to a 6mm top deflection (the maximum allowable storey drift of this structure under the serviceability limit according to Australian Standards (AS/NZS 1170 series)). The maximum tensile and compressive stresses shown in the graph are both around 62.7MPa , which are beyond the tensile and compressive capacity ranges of commonly used construction concrete for precast panels ($2\sim 5\text{MPa}$ for tensile strength and $20\sim 60\text{MPa}$ for compressive strength).



(a)



(b)

Figure 7-6 Stress distribution within the infill wall when subject to the maximum allowable storey drift according to Australian Standards (AS/NZS 1170) (Unit: Pa)

However, based on the field reconnaissance within the Asian-Pacific region, the extreme scenario depicted above seldom happens as some measures have been adopted in practice. In Australia, gaps between infill walls and the frame are specified in the structural design and are filled using elastic materials during construction. This, to some extent, reduces the chance of direct contact between infill walls and the structural frame, and thus provides a margin for the actual movement of the infill wall. Moreover, in some other countries, for example, China, masonry infill walls are built in the frame with the top layer bricks oriented along an in-plane 45 degree diagonal line (Figure 7-7). By doing this, the stress transferred from the frame to infill walls can be effectively dispelled.

Moreover, the out-of-plane behaviour of the infill panels such as the buckling issue have not been covered in this study. It is understandable that with a high slenderness ratio (in this study 30:1), the infill walls will tend to buckle under the combination of gravity loads and the out-of-plane loads. Under these circumstances, the contribution of infill walls to the structural stiffness will be diminished and the algorithm of the analysis needs to be revised. However, this study only focuses on the serviceability of the structure and the in-plane behaviour of infill walls rather than its out-of-plane behaviour. Taking into account the practices introduced above in different countries, the opportunity for the infill wall to buckle under this circumstance is slim. Thus, the out-of-plane behaviour of infill walls is beyond the scope of this study.

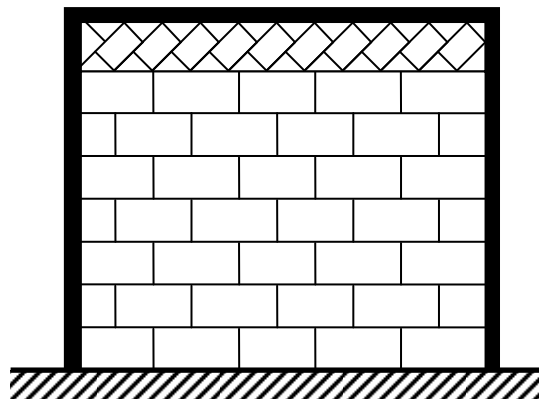


Figure 7-7 Simple demonstration of the practice used for dissipate the load transferred from the frame to infill walls in China

7.2.3.3 Coupled Shear Wall Analysis

Based on the theory from Stafford-Smith and Coull (1991) (Equations 7-12 to 7-22), a coupled shear wall under lateral loads acts as a pure cantilever. Figure 7-8 show the

results from both the theoretical and finite element analyses of the coupled shear wall. When under 120kN lateral load, top deflections of the coupled shear wall under theoretical and finite element analyses are 13.7mm and 14.7mm respectively. The difference between those two sets of results is less than 7%, which validates the finite element coupled shear wall model involved in this study.

$$y = \frac{wH^4}{EI} \left[\begin{array}{l} \frac{1}{24} \left(\left(1 - \frac{z}{H} \right)^4 + 4 \times \frac{z}{H} - 1 \right) + \\ \left(\frac{1}{2 \times (k\alpha H)^2} \right) \times \left(2 \times \frac{z}{H} - \left(\frac{z}{H} \right)^2 \right) - \\ \frac{1}{k^2} \left[\frac{1}{24} \times \left(\left(1 - \frac{z}{H} \right)^4 + 4 \times \frac{z}{H} - 1 \right) - \right. \\ \left. \frac{1}{(k\alpha H)^4 \cosh k\alpha H} \times (1 + k\alpha H \sinh k\alpha H - \cosh k\alpha z - k\alpha H \sinh k\alpha(H - z)) \right] \end{array} \right] \quad 7-12$$

$$y_{\max} = \frac{wH^4}{8EI} F_3(k, \alpha H) \quad 7-13$$

$$F_3 = 1 - \frac{1}{k^2} \left(1 - \frac{4}{(k\alpha H)^2} + \frac{8}{(k\alpha H)^4 \cosh k\alpha H} (1 + k\alpha H \sinh k\alpha H - \cosh k\alpha H) \right) \quad 7-14$$

$$k^2 = 1 + \frac{AI}{A_1 A_2 l^2} \quad 7-15$$

$$\alpha^2 = \frac{12I_c l^2}{b^3 h l} \quad 7-16$$

$$I_c = \frac{I_b}{1+r} \quad 7-17$$

$$r = \frac{12EI_b}{b^2 GA} \lambda \quad 7-18$$

$$\lambda = 1.2 \quad 7-19$$

$$I = I_1 + I_2 \quad 7-20$$

$$A = A_1 + A_2 \quad 7-21$$

$$G = \frac{E}{1+\gamma} \quad 7-22$$

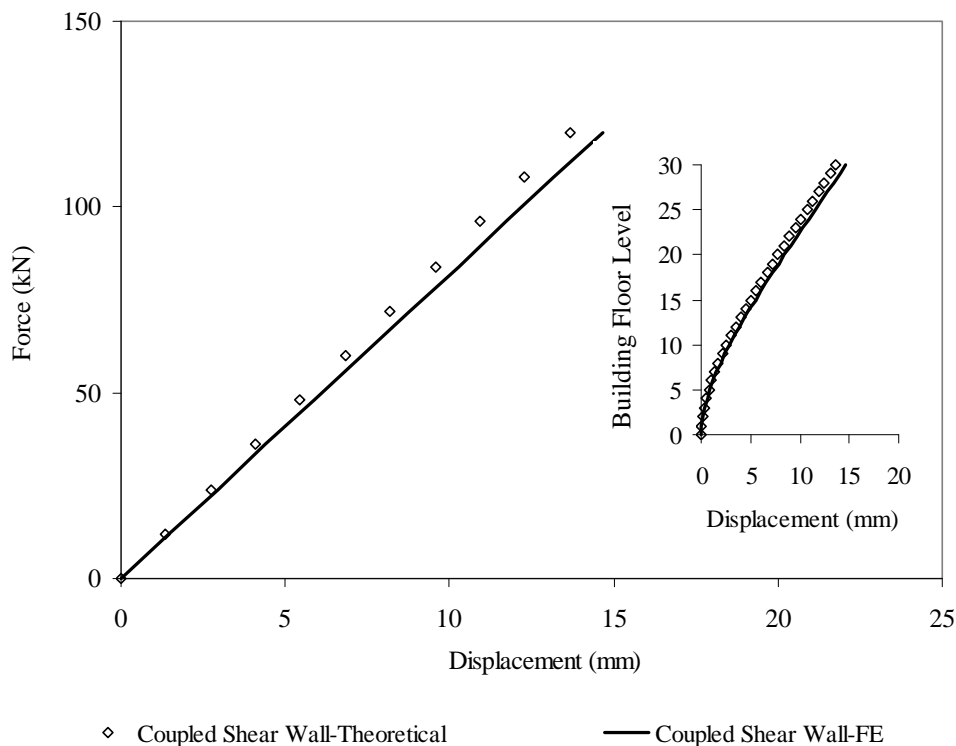


Figure 7-8 Force-displacement relationship and the total storey drift of coupled shear wall analysis

7.2.3.4 Coupled Shear Wall with Windows

Doors and windows are non-structural components similar to infill walls which can enhance the in-plane stiffness of the coupled shear wall. However, because of installation techniques and material properties, the bracing effect of doors and windows is not as quantifiable as that of infill walls. According to the reviewed literature mentioned in Chapter 2, limited information is available in detailing the performance of coupled shear walls with doors and windows. Consequently, only computer models were developed and analysed in this section.

The stiffening effect of windows

Figure 7-9 shows the comparison of the stiffness of a coupled shear wall with and without window panels. It is clear that even though it is not significant, windows in the coupled shear wall help reduce the storey drift of the structure by around 1.2%.

The stress distribution within the window panels

Though the stiffening effect of windows is not significant, it is still important to investigate the stress distribution within the window panels because the integration of window panels into the structural analysis will lead to the load redistribution within different elements, thus possibly overstress the non-structural components which had been considered as “non-load bearing components”.

Figure 7-10 plots the stress distribution within the window panel when it is subjected to the compliant drift of the allowable building storey drift according to Australian Standards (AS/NZS 1170 Series). From the graph, the maximum tensile stress in the glass panel is 1.65MPa . This is well within the range of the tensile capacity of the glass (27MPa to 62MPa).

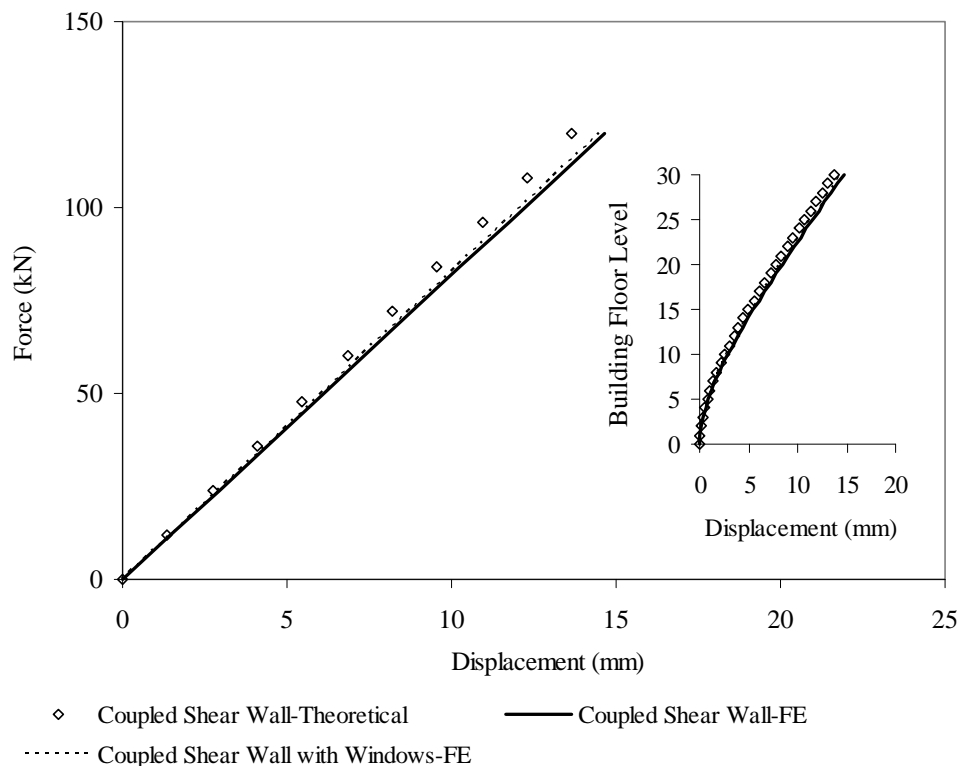


Figure 7-9 Comparison of the force-displacement relationship and the total storey drift of coupled shear wall with and without windows

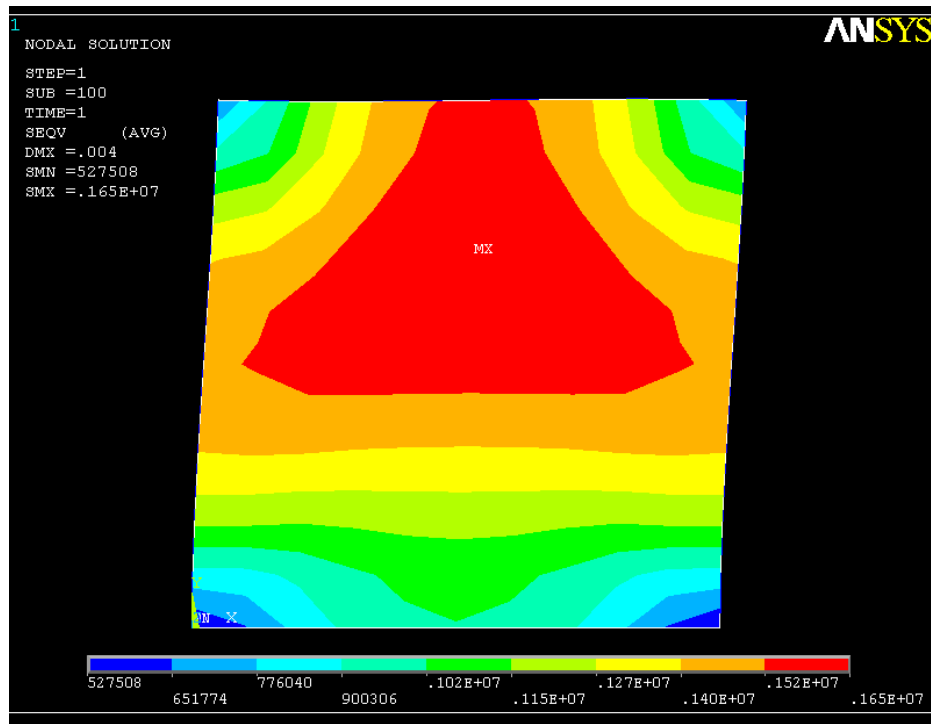


Figure 7-10 Stress distribution within the window panel when subject to the maximum allowable storey drift according to Australian Standards (AS/NZS 1170) (Unit: Pa)

7.2.3.5 Shear Wall Frame

This analysis sets the benchmark of the analysis of shear wall frames with façade panels. The theory adopted is from Stafford-Smith and Coull (1991). Based on the structural theory, the deflection of the wall-frame structure can be calculated by considering that the frame and walls are working together to resist the lateral loads (Equations 7-23 to 7-25)

$$y(z) = \frac{wH^4}{8EI} \left\{ \frac{8}{(\alpha H)^4} \left[\frac{\alpha H \sinh \alpha H + 1}{\cosh \alpha H} (\cosh \alpha z - 1) - \alpha H \sinh \alpha z + (\alpha H)^2 \left(\frac{z}{H} - \frac{1}{2} \left(\frac{z}{H} \right)^2 \right) \right] \right\} \quad 7-23$$

$$\alpha^2 = \frac{GA}{EI} \quad 7-24$$

$$GA = \frac{12E}{h_i \left(\frac{1}{G} + \frac{1}{C} \right)} = \frac{12E}{h \left(\frac{L_i}{\sum (I_g)_i} + \frac{h_i}{\sum (I_c)_i} \right)} \quad 7-25$$

Figure 7-11 shows the force-displacement relationships and the storey drifts of the shear wall frame from both theoretical and finite element analysis. The results are close to each other, with less than 10% difference.

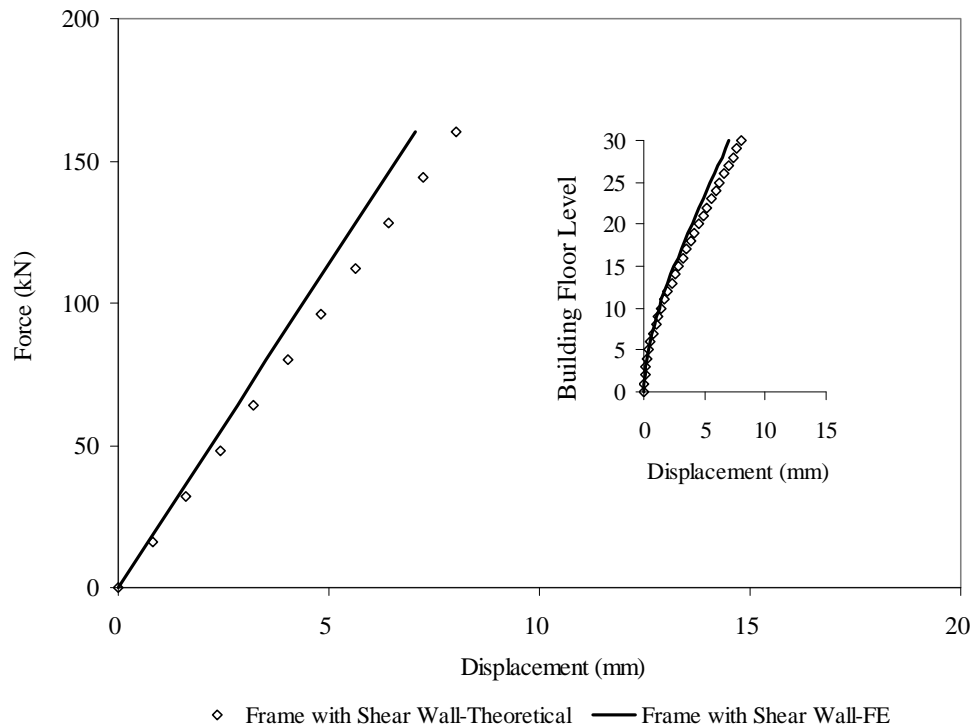


Figure 7-11 Force-displacement relationship and the total storey drift of frame with shear wall analysis

7.2.3.6 Shear Wall Frame with Façade Panels

In this analysis, the façade system is considered as a built-in façade, with glass panels framed by the aluminium mullions and jambs. Other types of façade system were also analysed, and the results are presented in the parametric study in this chapter.

According to Hoenderkamp and Snijder (2000; 2003), shear wall frames with façade systems can be considered to have an outrigger system (Figure 7-12). Based on the theory of outrigger systems developed by Stafford-Smith and Coull (1991), Equations 7-26 to 7-35 can be adopted to evaluate the performance of multi-storey shear wall frame structures with multi-storey façade systems.

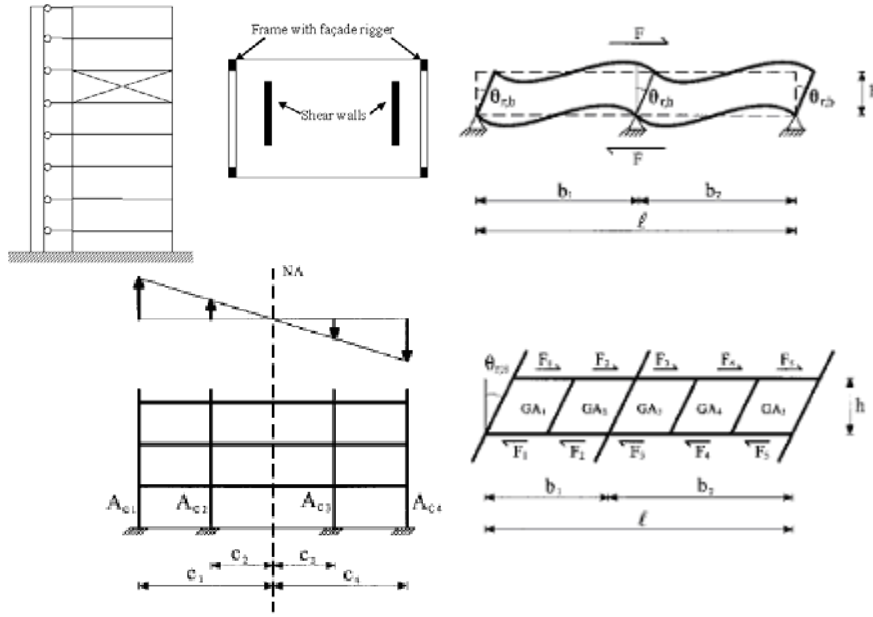


Figure 7-12 Shear wall frame with façade

$$\Delta_0 = \frac{wH^4}{8EI} - \frac{1}{2EI} \sum_{i=1}^n M_i (H^2 - x_i^2) \tag{7-26}$$

$$\begin{bmatrix} M_1 \\ M_2 \\ \vdots \\ M_i \\ \vdots \\ M_n \end{bmatrix} = \frac{w}{6EI} \begin{bmatrix} S_1 + S(H - X_1) & S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ S(H - X_2) & S_1 + S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_i) & S(H - X_i) & \cdots & S_1 + S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_n) & S(H - X_n) & \cdots & S(H - X_n) & \cdots & S_1 + S(H - X_n) \end{bmatrix}^{-1} \begin{bmatrix} H^3 - X_1^3 \\ H^3 - X_2^3 \\ \vdots \\ H^3 - X_i^3 \\ \vdots \\ H^3 - X_n^3 \end{bmatrix} \tag{7-27}$$

$$S = \frac{1}{EI} + \frac{2}{d^2(EA)_c} \tag{7-28}$$

$$S_1 = \frac{d}{12(EI)_0} \tag{7-29}$$

$$y_{\max} = \frac{wH^4}{8EI_w} - \frac{w(H^3 - x^3)(H^2 - x^2)}{12EI_w} \left\{ \frac{H}{EI_w \{S(H - x) + HS_r\}} \right\} \tag{7-30}$$

$$M_i = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \left(\frac{1}{EI_w} + \frac{2}{EA_c \ell^2} \right) (H - x) + \left(\frac{\ell}{12EI_f} + \frac{1}{h_i GA} \right) \right\}^{-1} \tag{7-31}$$

$$\omega = \frac{S_r}{S} = \left\{ \frac{\ell}{EI_r} + \frac{1}{h_i GA_r} \right\} \left\{ \frac{H}{EI_w} + \frac{H}{EA_c^2} \right\}^{-1} \tag{7-32}$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \frac{H}{S(H-x) + HS_r} \right\} \quad 7-33$$

$$S = \frac{H}{EI_w} + \frac{H}{EA_c^2} \quad 7-34$$

$$S_r = \frac{\ell}{12EI_r} + \frac{1}{h_i GA_r} \quad 7-35$$

Figure 7-13 indicates the final results from both theoretical analyses and finite element analyses of the shear wall frame with the façade system. It is observed that the difference between the theoretical and finite element analysis results is up to 22% at a load of 160kN. Moreover, the structure under theoretical analysis seems less stiff than it is under finite element analysis. This can be explained by analysing the theory presented by Stafford-Smith and Coull (1991), which considers the shear wall frame with the façade as an outrigger system. In this case, shear stiffness from the façade and the capacity of resisting flexural deflection from the aluminium façade frame are not taken into account.

Hoenderkamp and Snijder (2000; 2003) did improve the theory for analysing a structure with a façade system. However, in reality, the complexity of different façade systems from connections to façade assemblies is a limitation that makes it extremely difficult to simply utilise one theory, especially for multi-storey buildings with multi-storey façade systems.

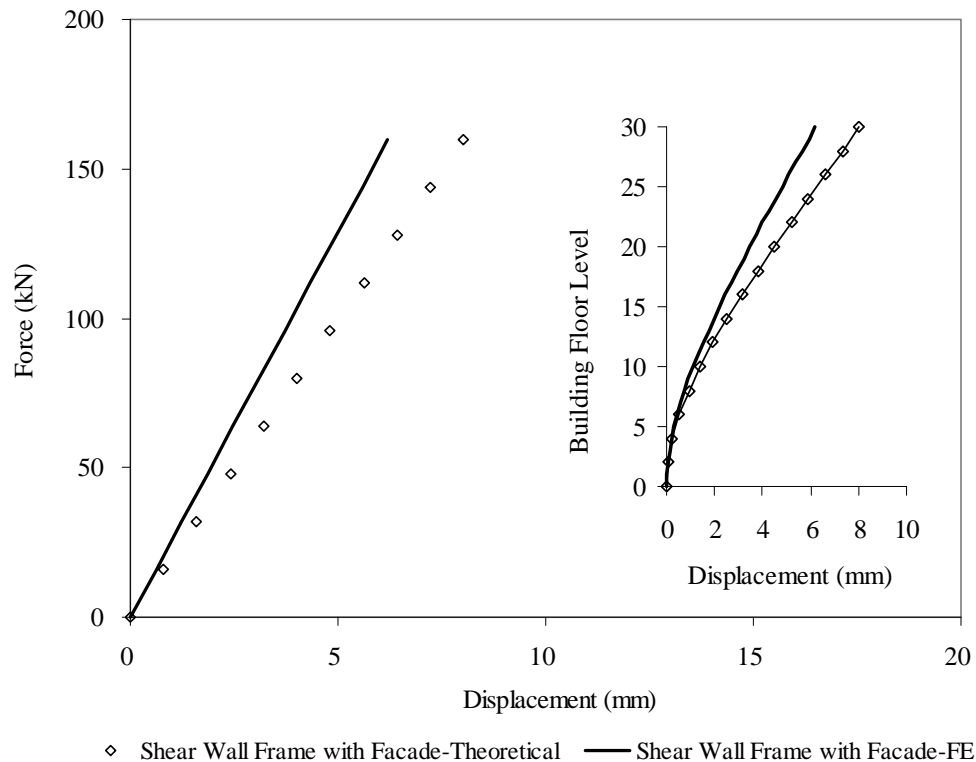


Figure 7-13 Force-displacement relationship and the total storey drift of shear wall frame with façade analysis

Stiffness contributions of the façade

Figure 7-14 compares the stiffness of shear wall frames with and without façades. According to the finite element results, it is clear that with the inclusion of the façade system into the structural analysis, a 12% increase of structural stiffness can be achieved. If compared with another type of façade system analysed in the parametric study (discussed in the following sections of this chapter), it is clear that with the variation of the façade type, the stiffening effect of the façade to the structural system also varies.

Stress distribution within façade panels

Similar to the analysis for the infill walls, maximum allowable deflection (according to the Australian Standards, (AS/NZS 1170 series)) was applied to the façade panel so that the stress distribution within the panel can be evaluated. Figure 7-15 shows the maximum tensile stress within the façade panel. It is similar to that of the window panels, even though the tensile stress is as high as 1.6MPa , it is still well within the tensile capacity of glass, which is from 27MPa to 62MPa .

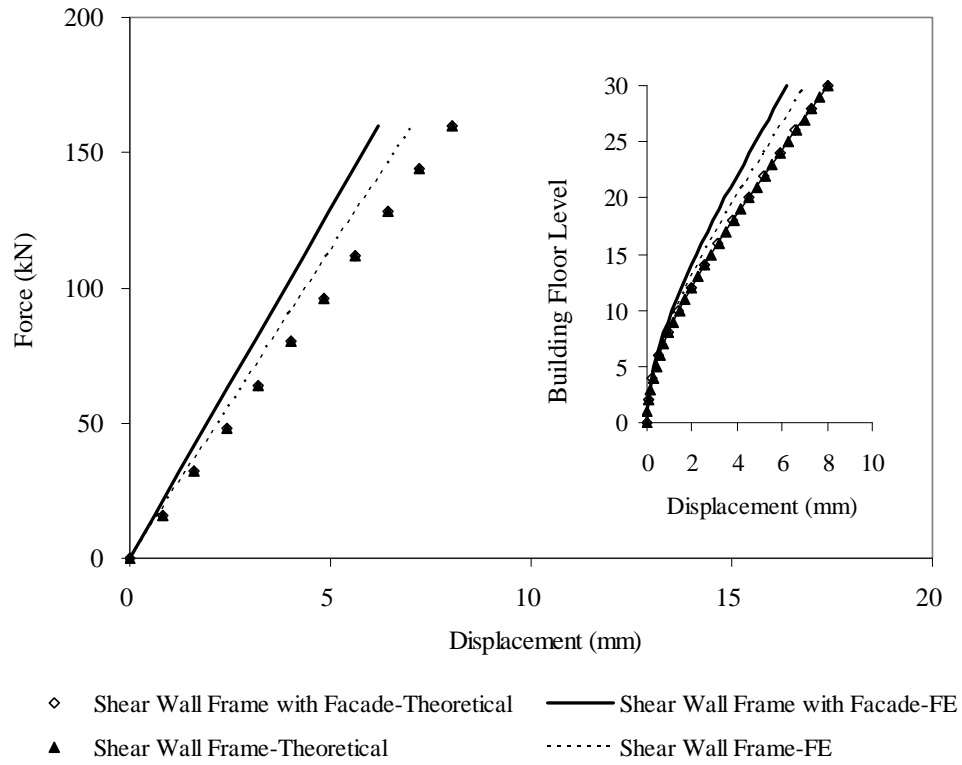


Figure 7-14 Comparison of the force-displacement relationship and the total storey drift of shear wall frame with and without façade

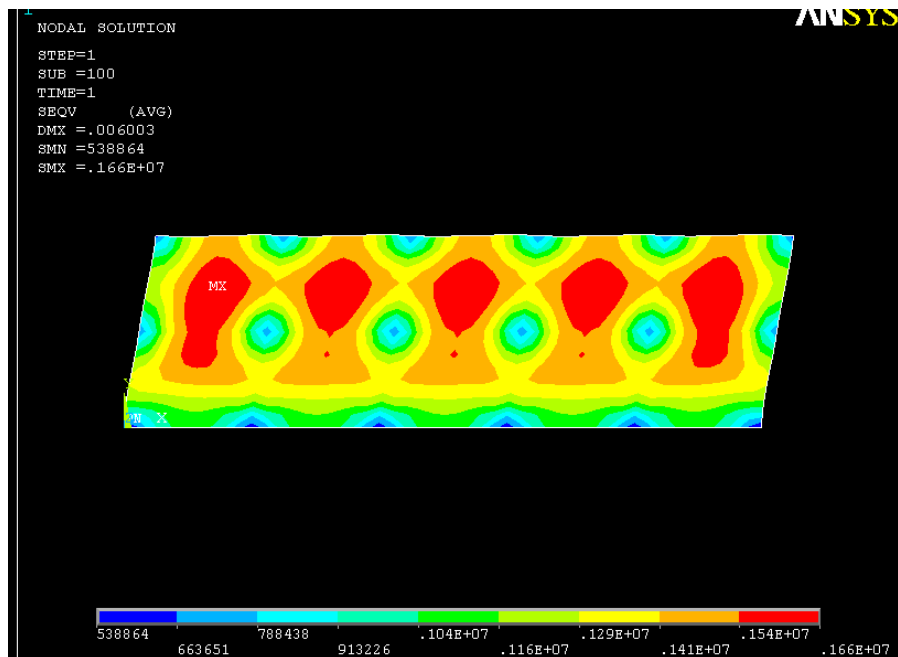


Figure 7-15 Stress distribution within the façade panel when subject to the maximum allowable storey drift according to Australian Standards (AS/NZS 1170) (Unit: Pa)

7.2.4 Conclusions of the Theoretical Analysis

This study analysed the performance of different combinations of structure and non-structural components. The stiffness contributions of different components were also identified. Based on the analysis, the following conclusions can be drawn:

- Even though identified as non-structural components, infill walls and façade systems are very important in increasing structural stiffness;
- More than a 60% extra stiffness contribution could be made by the infill walls to the lateral load resisting system of the structure, based on the structures presented in this study;
- With the structural frame considered in this paper, if the façade system is a built-in façade, the stiffness of the shear wall frames with façades will be 12% higher than those without façades;
- Even though the stiffening effect is not as significant as that of infill walls, when considering the serviceability of buildings, windows also have slight contribution (approximately 1.2%) to the structural stiffness of the coupled shear wall;
- The type of façade system influences the overall stiffness of the structure. From a built-in façade (i.e. built-in façade with aluminium mullions) to an off-set façade (parametric study), the difference of structure stiffness varies by around 0~16%
- Even though significant stiffness contributions can be realised by including infill walls into the structural analysis, in the worst scenario, which means direct contact between the infill wall and the structural frame happens all the time during the maximum service level movement of the building, the concrete block infill walls may not be able to withstand the storey drift under the serviceability limit set by the Australian Standards;
- Window panels and façade panels are capable of adapting to the compliant drift under the serviceability allowances of Australian Standards.

Based on the analyses carried out in this study, it is necessary that simplified but equivalent finite element models should be developed so that a parametric study on the influences of both individual elements and overall non-structural components can be evaluated, to form a basis on which design recommendations could be made.

7.3 A Parametric Study

The parametric study provides an overview of the design philosophy of high-rise buildings, by adopting details of a case-study structure and the results from a detailed theoretical analysis (provided in previous sections). It developed a series of finite element models to identify the influence of the non-structural components on the overall structural performance. Parameters representing non-structural components were varied in turn, to replicate the reasonable variation which might be found in current Australian practice.

Storey drift of a multi-storey building is a critical parameter in the serviceability design. In the mean time, moment, shear, and the combined capacities are key considerations in design of the ultimate strength. Any changes of the distribution of bending moment and shear force will cause dramatic variations of the strength design. Further more, the change of the bending moment and shear force in the structural elements will also influence the storey drift significantly. According to Taranath (1998), the total deflection of the normally proportioned rigid frame can be roughly regarded as a combination of the following four factors:

- Deflection due to the axial deformation of columns (15% ~ 20%);
- Frame racking due to beam rotation (50% ~ 60%);
- Frame racking due to column rotation (15% ~20%);
- Deflection due to joint deformation (very small).

In this section, the above influencing factors were re-categorised into two main streams: the flexural performance and the shear behaviour. The flexural performance of the structure can be expressed by the bending moment distributed along columns and the rotation of the joints. Similarly, the performance under shear forces of the structure can be represented by the shear force distribution in the columns. Assume that joints of elements are all rigid. The deformation of joints is discarded in the discussion of this section.

The objective of this study is to quantify the influence of non-structural components on the overall stiffness, flexural, shear, and the rotational behaviour of a case study structure. Detailed analyses of storey drift, natural frequency, shear force and bending

moment distributions and joint rotations of the structure under different structural configurations were conducted.

7.3.1 Outline of the Study

A series of finite element models were developed to represent different assemblies of elements in a typical tall building. They are:

- The skeleton frame;
- The frame with service cores;
- The core-frame with infill walls;
- The core-frame with shear walls;
- The core-frame with shear walls and façade.

In the analysis of the storey drift for each model, theoretical verifications are also provided.

At each stage, the following scenarios were discussed:

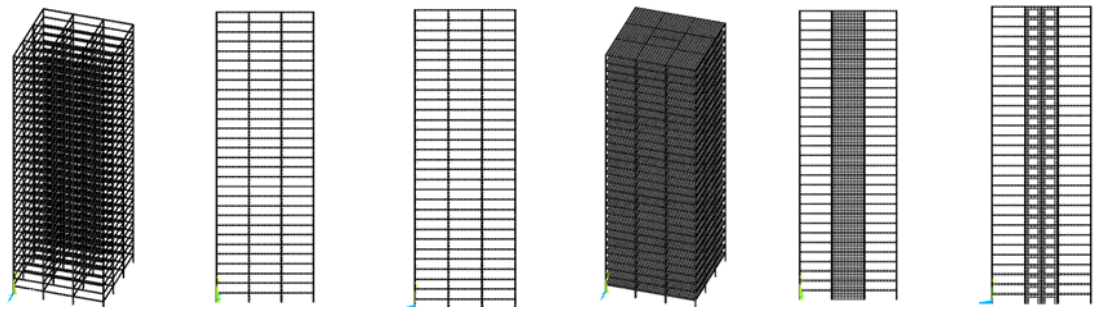
- Influence of the non-structural components on the serviceability of the structure. In this section, the contribution of non-structural components to the storey drift was quantified; modal analysis was also conducted to evaluate the influence of non-structural components to the natural frequency of the structure;
- Influence of the non-structural components to the flexural performance of the structure. In this part, the distributions of the bending moment of the outer column of the structure under different structure configurations were compared;
- Influence of the non-structural components to the shear performance of the structure. Similar to the analyses of the flexural contributions of non-structural components, the shear force distributions of the outer columns of the models were plotted and discussed;
- Influence of the non-structural components to the joint rotation of the structure. Since the joint rotation has direct relationship to the deflection, it would be more

convincible/self-explanatory if the rotational behaviour of the elements can be explored and compared with the deflection obtained from the same analysis.

A symmetric structural frame was adopted, with the dimension of $30m$ by $30m$ by $90m$. The base-to-height ratio of the structure therefore is 1:3. The floor plan is divided into 9 bays by columns and beams. The storey height is $3m$. (Figure 7-16(f))

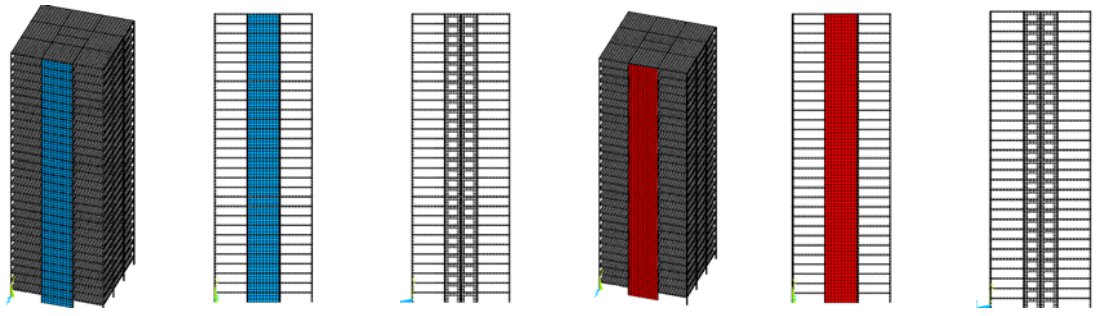
7.3.1.1 Geometric and Material Properties

To simplify the analysis, a uniform cross section steel beam 360UB56.7 was assigned to the beams and a steel 310UC118 column to columns (Figure 7-16(a)). The service core is composed of four shear walls: two with openings (coupled shear walls) and two without openings (Figure 7-16(b)). The thickness of these four shear walls is $0.25m$. Concrete block infill walls were included in the two parallel central bays of the frame (Figure 7-16(c)). Very small gaps between infill walls and the surrounding frame elements were defined to simulate the installation of infill walls in the practice (Figure 7-17). Two parallel single-bay concrete shear walls were installed on the frame in the shear wall frame structure analysis. Each of the shear walls has the thickness of $0.4m$ (Figure 7-16(d)). The façade system is comprised of an aluminium frame and glass panels (Figure 7-16(d)). The cross sectional area of the aluminium frame is $0.05m$ by $0.05m$. The distance between mullions is $2.5m$. The thickness of the glass panels is $0.02m$. Details of the geometric and material properties of these structural and non-structural components are listed in Table 7-2.



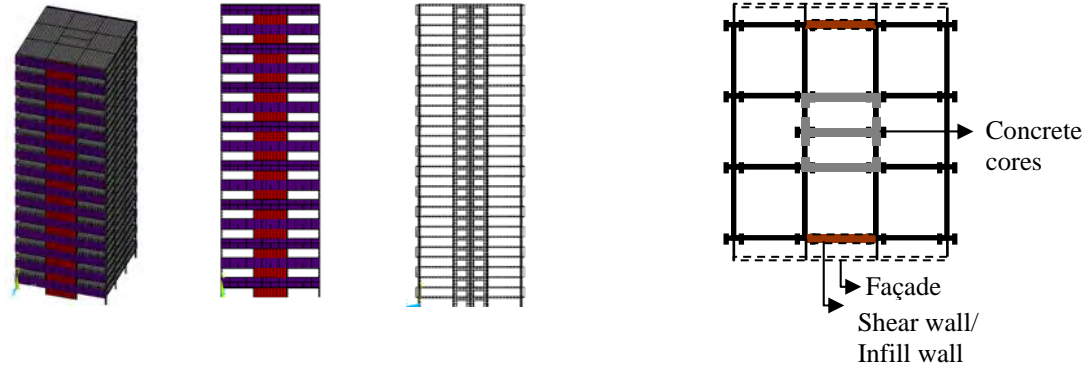
3-D view Front view Right view 3-D view Front view Right view

(a) Skeleton Frame (b) Frame with Service Core



3-D view Front view Right view 3-D view Front view Right view

(c) Core-frame with Infill Walls (d) Core-frame with Shear Walls



3-D view Front view Right view

(e) Core frame with shear wall and façades (f) Floor plan

Figure 7-16 Structural plan

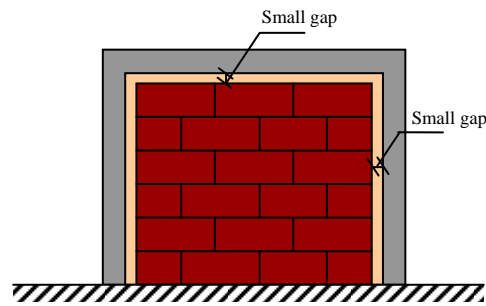


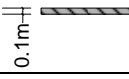
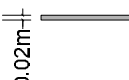
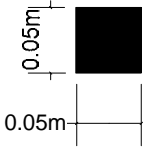


Figure 7-17 Infill walls

Table 7-2 Details of elements and materials

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column	360UB56.7	Steel	Linear Elastic $E = 2.0 \times 10^{11} Pa$	BEAM4
Beam	310UC118		$\mu = 0.29$ $density = 7850 kg/m^3$	
Shear Core		Concrete	Linear Elastic $E = 2.5 \times 10^{10} Pa$ $\mu = 0.15$ $density = 2400 kg/m^3$	SHELL63
Shear Wall				
Infill Wall				
Façade Panel		Glass	Viscoelastic $G_0 = 2.74 \times 10^{10} Pa$ $G_b = 6.05 \times 10^{10} Pa$ $1/\beta = 0.53$ $density = 2390 kg/m^3$	SHELL63
Façade Frame			Aluminium	
Contact	--	--	--	CONTAC52

7.3.1.2 ANSYS Elements and Boundary Conditions

A 3-D beam element was used to define the beams and columns of the structural frame, and the frame of the façade system. Shear walls, infill walls and façade glass were represented by 3-D shell elements. The possible contact between infill walls and the structural frame was defined by contact pairs that included both target and contact elements. Details are in Table 7-2.

All 6 degrees of freedom of the base of the structure were constrained to simulate a fixed condition and to eliminate the influence of the foundation. The structural system is symmetric and so is the lateral loading condition. A 0.4KPa lateral load was incorporated in the model as a series of equivalent point loads applied to the beam/column joints up one side of the model, as shown in Figure 7-18.

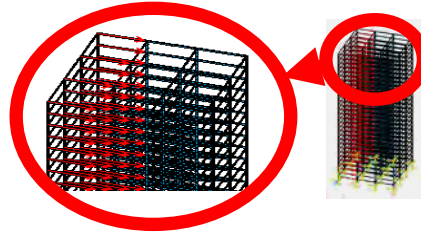


Figure 7-18 Boundary conditions of the model

7.3.2 Analysis of the Influence of the Non-structural Components on the Serviceability of the Structure

In this section, both static and dynamic characteristics were investigated; focusing on the differences caused by the non-structural components. Storey drifts and natural frequencies are the main foci of the comparison.

7.3.2.1 Theories for the Analyses

As discussed in the previous sections, theoretically, the top deflection of a multi-storey frame can be considered as an accumulation of the total storey drift which is the sum of storey drifts caused by column flexure, girder flexure, and storey drift due to overall bending. This is represented by Equations 7-1 to 7-6. (Stafford Smith and Coull 1991).

Based on the structural theory, the deflection of the wall-frame structure can be calculated by considering that the frame and walls are working together to resist the lateral loads. Formulae were given by Stafford-Smith and Coull (1991) as Equations 7-23 to 7-25.

Infill walls are always considered as non-structural elements providing bracing effects to the structural frame. Thus, typical theory for infilled-frame structures represents the infill walls by using equivalent bracing elements. The drift in storey i , is a combination of deflection caused by the shear deflection of braced bents at storey i and the total storey drift due to bending (refer Equations 7-7 to 7-11).

According to Hoenderkamp and Snijder (2000; 2003), a shear wall structure with a façade system can be considered as an outrigger system. Based on the theory of outrigger systems developed by Stafford-Smith and Coull (1991), equations 7-26 to 7-35 can be employed to evaluate the performance of multi-storey shear wall frame structures with multi-storey façade system. The connections between the façade system and the structural frame were considered as rigid in this analysis. The influence of the connection properties are discussed in Chapter 5.

7.3.2.2 Results and Discussions

Figures 7-19(a) to (e) compare the storey drifts obtained from both the theoretical and finite element analysis of the different structural configurations. There is close correlation between the theoretical and finite element analysis results for all the models for each configuration.

Figure 7-20(a) compares the storey drifts of different assemblies of elements under the same loading condition. In terms of the contributions of different non-structural components, according to the results presented in Figure 7-20(b), under the same lateral loading condition, a reduction of approximately constant 9% in the deflection can be achieved by including two parallel single-bay multi-storey infill walls to the structural analysis (as shown in Figure 7-16). The core-frame structure has a deflection at the top of the building of 22mm. However, including shear walls in the frame structure reduces the deflection by more than 27%. In taking a further step of including façade panels in the shear wall frame, it is seen from Figure 7-20(b) that the contribution of façade to the reduction of the structural deflection decreases with the increase of the building height, from around 16% at the bottom to less than 0.1% at the top.

Table 7-3 and Table 7-4 list natural frequencies of the first 3 modes of the different models with and without non-structural components in the direction orthogonal to the loading direction. In Table 7-3, it is clear that there is no obvious change to the first mode frequency for the skeletal structure with different configurations since the non-structural components were not included in the direction orthogonal to the loading direction. However, by checking the second mode and the third mode (twisting mode) frequencies, obvious changes appear in different structural configurations. By including shear walls to the core-frame structure, the frequencies of the second and third modes of the structure increase more than 20% and 21% respectively. Approximately 5% and 9% increases to the second and third modes frequencies are induced by adding infill walls to

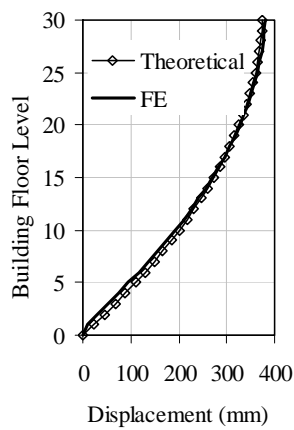
the structure. This means that if the non-structural components are not included in the direction orthogonal to the loading direction, there will be no significant contribution to the fundamental frequency. However, for the second mode and the twisting mode frequencies, the significant influence of infill walls can be identified whilst there are minor contributions from façade panels. Table 7-4 shows how the fundamental frequencies of the structure are changed by including different non-structural components orthogonal to the direction of the load. The change caused by adding infill walls to the orthogonal direction of loads is more than 7% whilst façade panels only have slight influence on the fundamental frequency of the structure (less than 1%).

Table 7-3 Frequencies of structures under different modes (without the inclusion of non-structural components in the direction orthogonal to the loading direction)

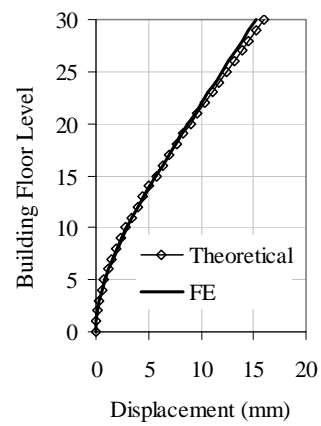
Structural Configuration	Mode 1	Mode 2	Mode 3
	Frequency Hz	Frequency Hz	Frequency Hz
Frame	0.06	0.07	0.12
Frame + Core	0.30	0.32	0.54
Frequency Change by Core (%)	403.83%	385.97%	364.47%
Frame + Core Infill Wall	0.30	0.34	0.59
Frequency Change by Infill Walls (%)	0.48%	4.93%	9.02%
Frame + Core + Shear	0.32	0.39	0.65
Frequency Change by Shear Walls (%)	6.54%	20.97%	21.86%
Frame + Core Shear + Façade	0.32	0.39	0.63
Frequency Change by Façades (%)	-0.13%	-1.38%	-3.52%

Table 7-4 Frequencies of structures under different modes (with the inclusion of non-structural components in the direction orthogonal to the loading direction)

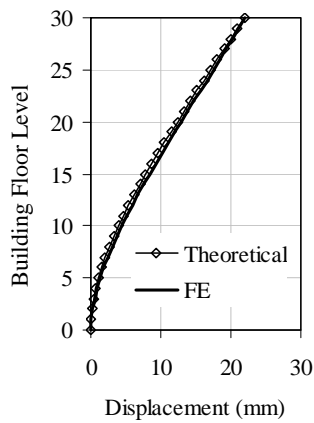
Structural Configuration	Mode 1	Mode 2	Mode 3
	Frequency Hz	Frequency Hz	Frequency Hz
Frame	0.06	0.07	0.12
Frame + Core	0.30	0.32	0.54
Frequency Change by Core (%)	403.83%	385.97%	364.47%
Frame + Core Infill Wall	0.32	0.34	0.62
Frequency Change by Infill Walls (%)	7.46%	5.99%	15.00%
Frame + Core + Shear	0.39	0.42	0.70
Frequency Change by Shear Walls (%)	28.57%	29.00%	31.29%
Frame + Core Shear + Façade	0.39	0.42	0.70
Frequency Change by Façades (%)	0.56%	0.49%	-1.33%



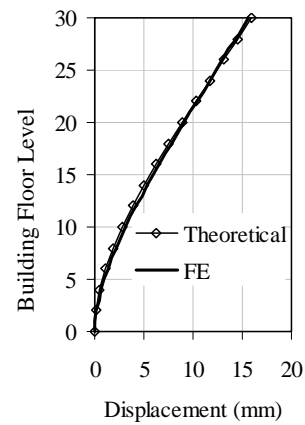
(a) Skeleton frame



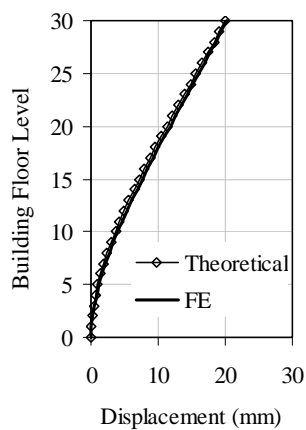
(d) Core-frame with shear walls



(b) Frame with shear cores

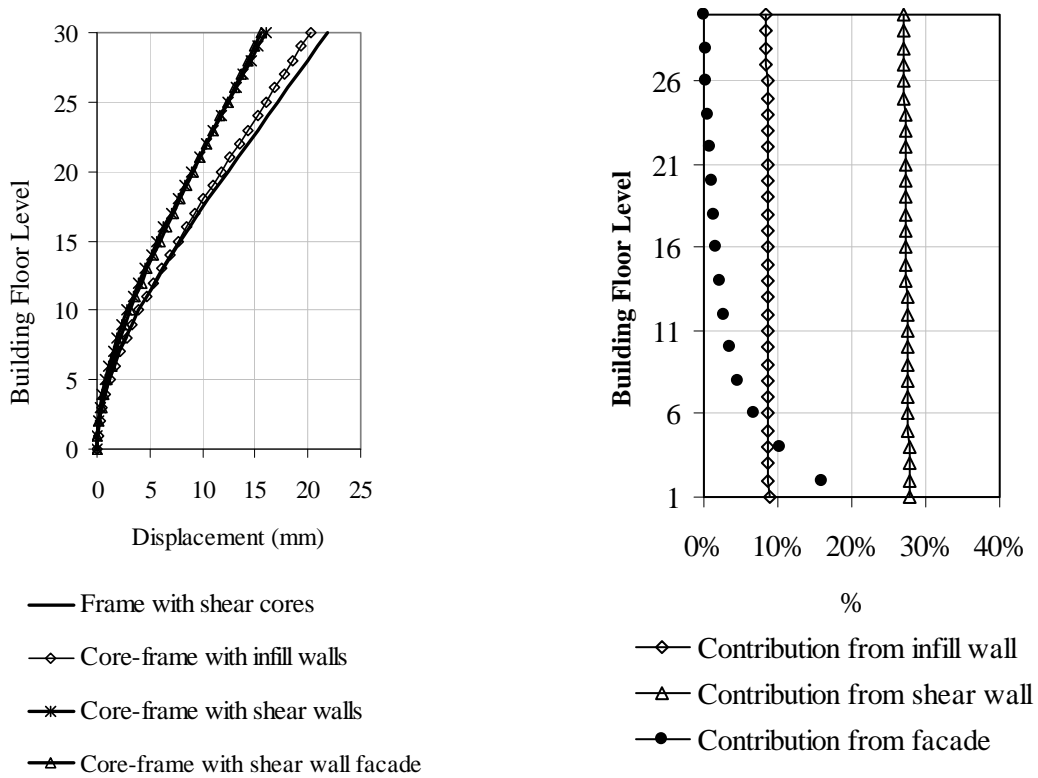


(e) Core-frame with shear wall façades



(c) Core-frame with infill walls

Figure 7-19 Storey drift of assemblies with different structural and non-structural components (Lateral loading: 0.4KPa)



(a) Storey drift

(b) Contribution of different components

Figure 7-20 Comparison of the storey drift contributions from different components
(Lateral loading: 0.4KPa)

7.3.3 Analysis of the Influence of Non-structural Components to the Flexural, Shear, and Rotational Performance of the Structure

Based on the theory from Taranath (1998), the total deflection of the normally proportioned rigid frame can be roughly considered as a combination of the following four factors:

- Deflection due to the axial deformation of columns (15% ~ 20%);
- Frame racking due to beam rotation (50% ~ 60%);
- Frame racking due to column rotation (15% ~20%);
- Deflection due to joint deformation (very small).

In this section, the flexural performance and shear performance of the structural frame with different non-structural components were analysed, the bending moment and shear force distributions on both inner and outer columns were investigated (The specific

outer and inner columns are identified in Figure 7-21 (a) to (d) for each type of the model).

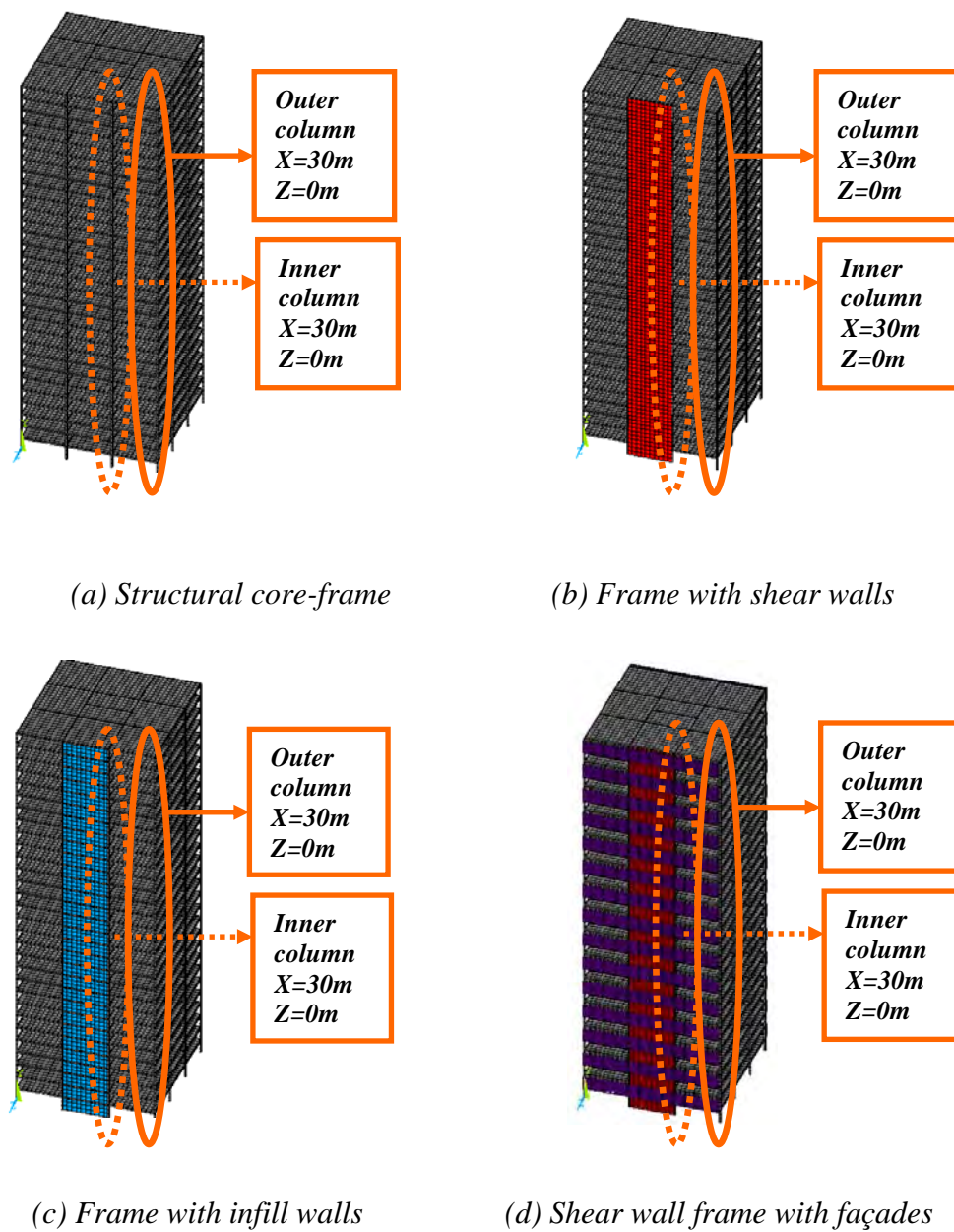


Figure 7-21 Identification of inner and outer columns in different structural configurations

7.3.3.1 Influence of Non-structural Components to the Flexural Performance of the Structure

The deflection of the rigid frame tall building structure is composed of two components: the cantilever bending component and the shear racking component (Taranath 1998).

Taranath also pointed out that the cantilever bending is mainly caused by the column deformation and it can contribute to 15 ~ 20% of the total deflection of the tall building.

In this study, the bending moment along the outer columns of the structural frame (leeward columns) was investigated, so that the influence of non-structural components to the flexural performance of the structure can be observed.

The bending moments distributed in the outer columns of structural frame with and without infill walls are shown in Figure 7-22. From the graph, a major 20% increase of bending moment in the outer columns is caused by including infill walls to the structural core-frame.

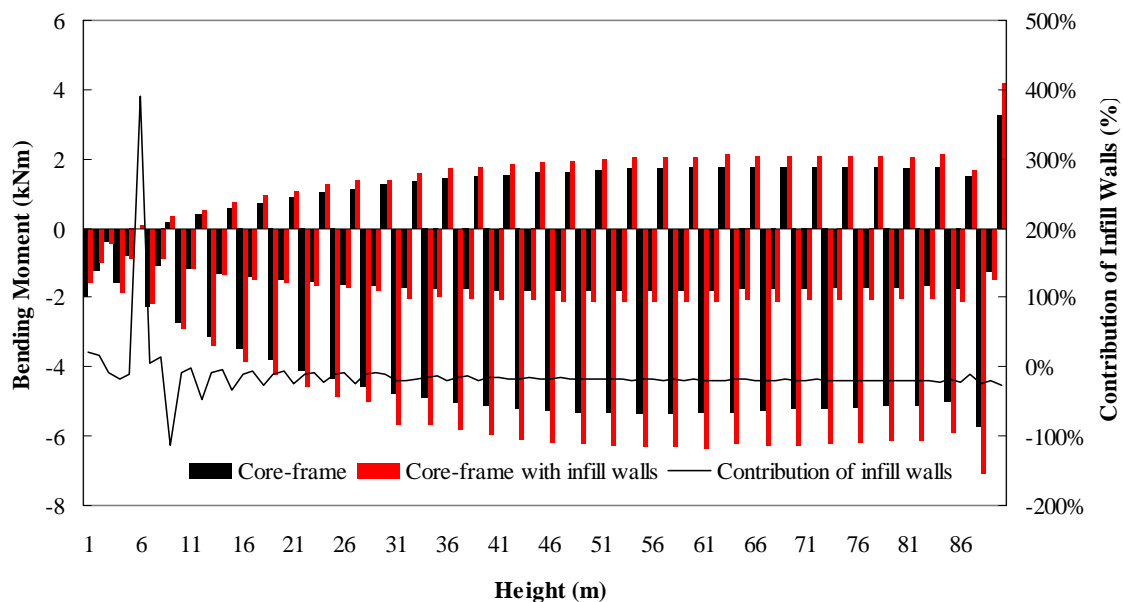


Figure 7-22 Bending moment distribution in the outer columns of structural core-frame with and without infill walls

The bending moments in the inner columns of the structural frame with and without infill walls are plotted in Figure 7-23. It can identify that an average of 25% decrease of bending moment in the structural inner columns can be achieved by including infill walls to the analysis.

Moreover, by comparing Figure 7-22 and 7-23, the influence of infill walls to the bending moment distribution in the outer columns tends to decrease dramatically at level 2 of the building. However, remarkable decrease of the bending moment in the inner columns can be observed at the same level. With the further investigation of the

results, it is mainly because of the changing of contra-flexural points caused by adding infill walls to the structural frame.

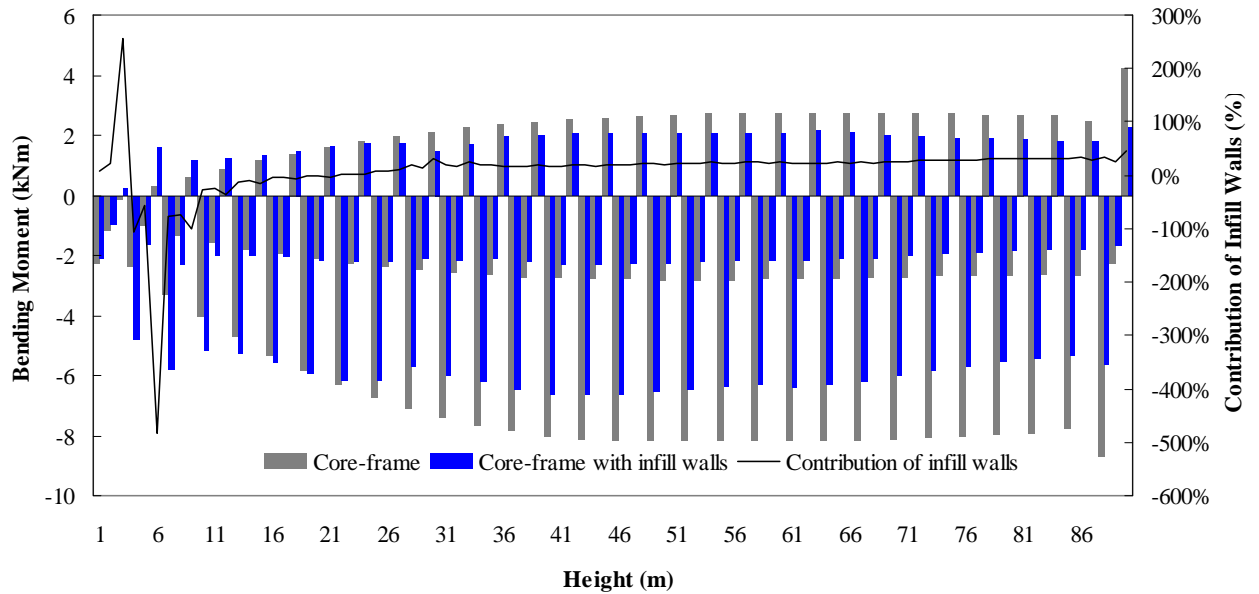


Figure 7-23 Bending moment distribution in the inner columns of structural core-frame with and without infill walls

Figure 7-24 and 7-25 compare the bending moment distributed in the outer and inner columns of the structural frame with and without shear walls. The bending moment along the inner columns is significantly reduced because of the inclusion of shear walls. To be specific, the bending moment along the inner columns of the structure is reduced by more than 120% (comparing with the absolute value of the response) by adding shear walls to the structural core-frame. However, only slight changes (4%) happen to the outer columns when including shear walls in the structural analysis.

Similar to the infill walls, because of the changing of contra-flexural point, significant changes of the bending moment in the outer and inner columns of the structural frame happen at the second storey

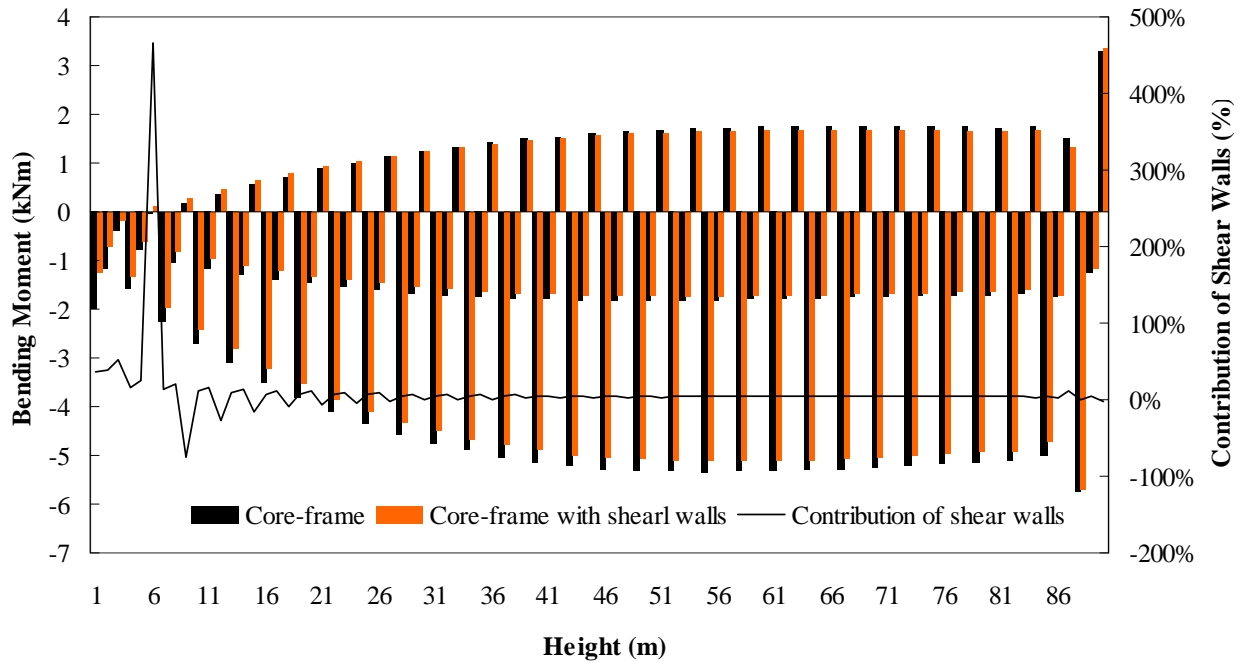


Figure 7-24 Bending moment distribution in the outer columns of structural core-frame with and without shear walls

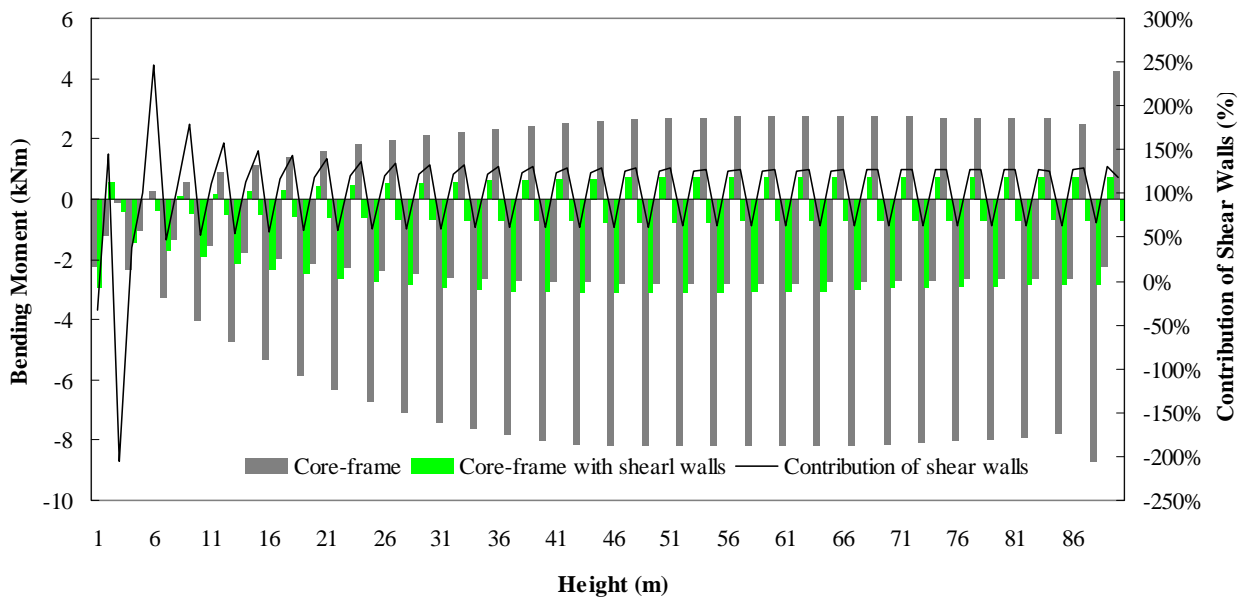


Figure 7-25 Bending moment distribution in the inner columns of structural core-frame with and without shear walls

Shear wall frame structures with and without façade systems were analysed and the results are plotted in Figure 7-26 and 7-27. From Figure 7-26, only around 2% increase of the bending moment in the outer columns can be realised by installing façade panels to the main structural frame. Similarly, regarding the bending moment in the inner columns, Figure 7-27 indicates that around 2% increase of the bending moment also happens to the inner columns by including façade system to the structural frame.

There are obvious sharp break-outs in each graph (Figure 7-22 to 7-27). By checking details of each figure (Figure 7-22 to 7-27), it can be observed that those sudden changes of the bending moments happen near to the contra-flexural points of the columns. It means that the inclusion of the non-structural components in the structural analysis changes the locations of those contra-flexural points in the columns.

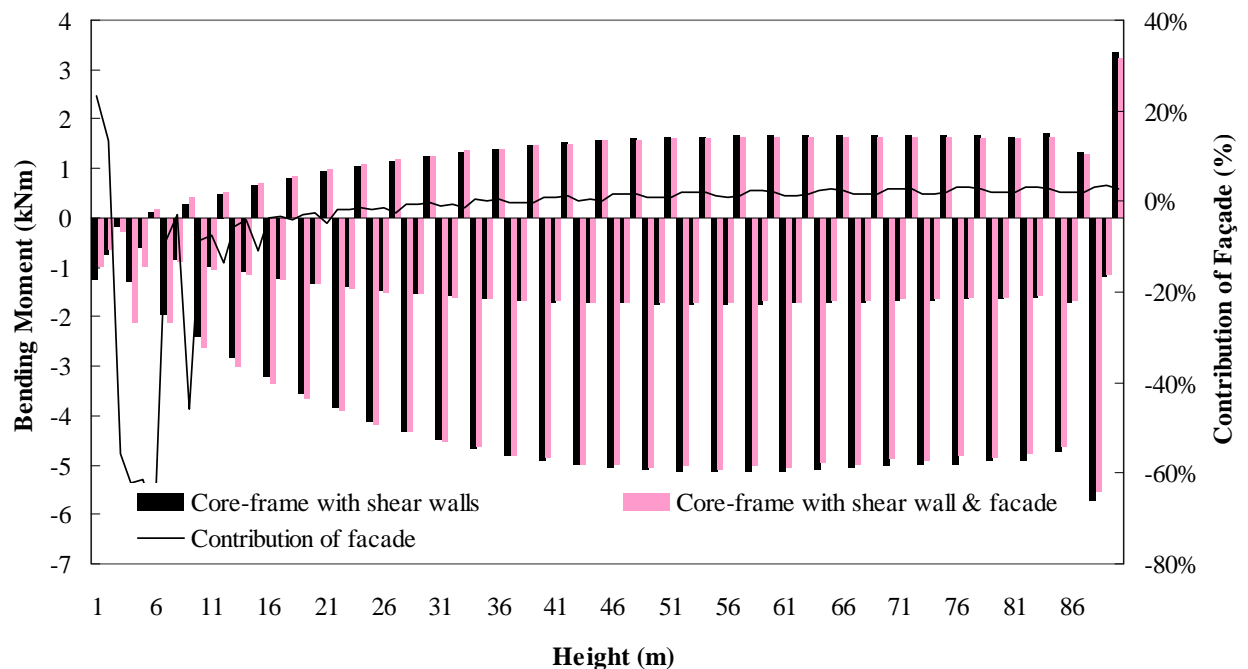


Figure 7-26 Bending moment distribution in the outer columns of structural core-frame with and without symmetric shear-wall façade system

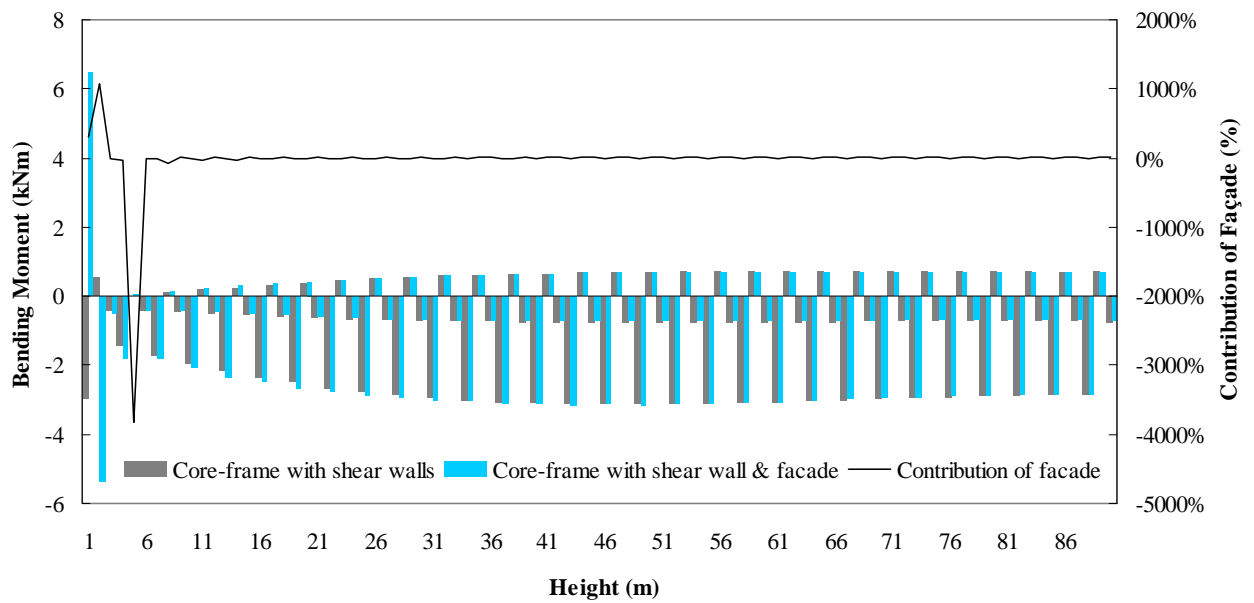


Figure 7-27 Bending moment distribution in the inner columns of structural core-frame with and without symmetric shear-wall façade system

7.3.3.2 Influence of Non-structural Components to the Shear Performance of the Structure

According to Taranath (1998), shear racking of the structure is caused by the deformation of beams and columns. By resisting the shear forces in each floor, the columns bend in double curvature with the contra-flexural point at their middle length. The moment at the joints from the columns is resisted by the beams, which also bend in double curvature. This mode of deformation can count for up to 80% of the total deflection of the structure.

In this study, shear forces under different structural configurations in the outer and inner columns of the frame are plotted, so that the influence of non-structural components can be identified.

The shear force distributions of the outer columns of the structural frame with and without infill walls are shown in Figure 7-28. It can be observed that more than 13% constant increase of shear force along the structural outer columns is realised by adding infill walls to the structural frame.

Figure 7-29 plots the shear force distributed in the inner columns of the structural frame with and without infill walls. Compared with the outer columns, shear forces along the inner columns opt to increase at the bottom half (from level 0 to level 21) whilst

decrease at the top half of the structure (from level 22 upwards). The maximum decrease and increase of shear forces (comparing with the absolute value of the response) are both more than 400%.

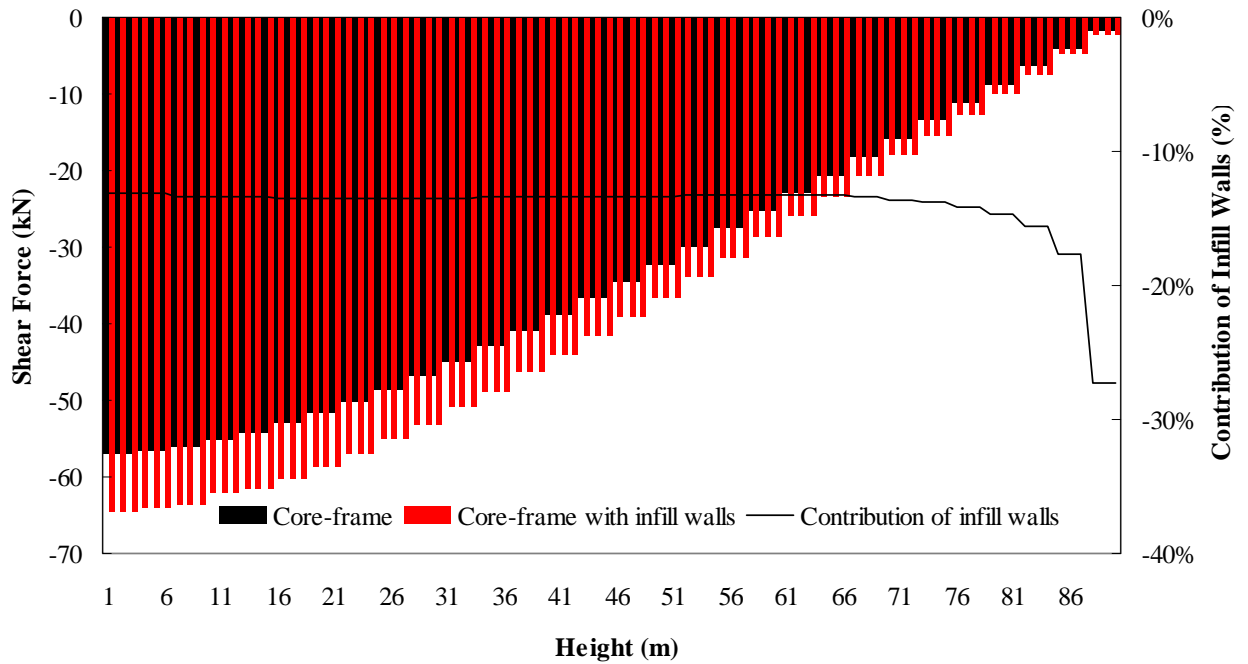


Figure 7-28 Shear force distribution in the outer columns of structural core-frame with and without infill walls

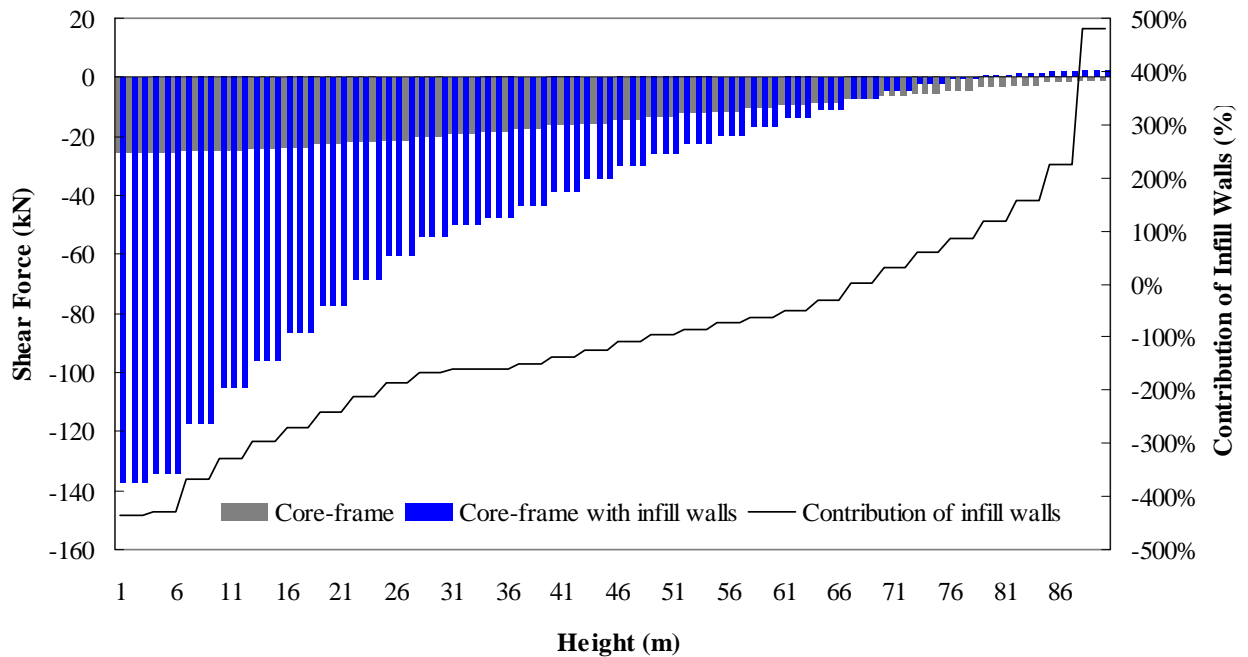


Figure 7-29 Shear force distribution in the inner columns of structural core-frame with and without infill walls

The contribution of shear walls to the shear force distribution in the outer columns of structural frame is plotted in Figure 7-30. Only subtle changes (3% of decrease) of the shear forces can be achieved by including shear walls to the structural core-frame.

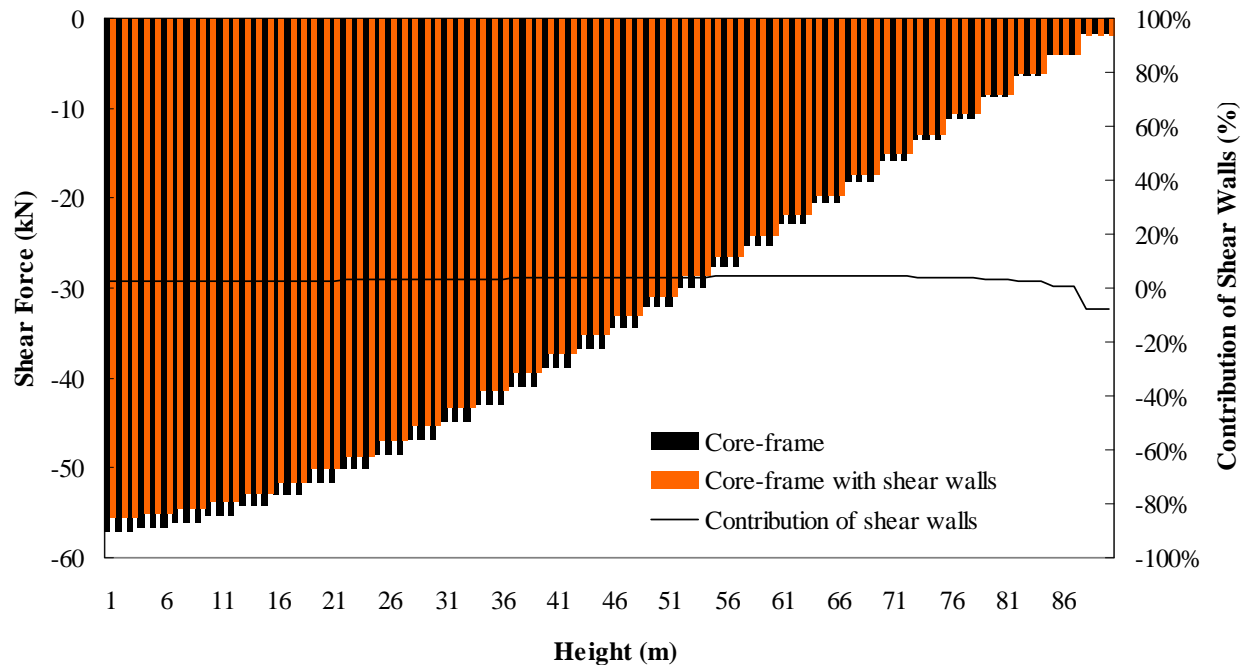


Figure 7-30 Shear force distribution in the outer columns of structural core-frame with and without shear walls

By checking Figure 7-31, significant influence of shear walls to the inner columns of the structural frame can be observed. If compared with the core-frame structure, because of the inclusion of shear walls, the shear forces in the inner columns increase at the bottom half of the structure from level 0 to level 17. The maximum amount of increment (400%, compared with the absolute value) happens at the base level of the structure. From level 18 upwards, the shear force starts to decrease, from 0% to more than 600% (of the absolute value of the original shear force).

By comparing Figure 7-30 and 7-31, it can be identified that with the increase of the height, shear forces of the core-frame with shear wall structure decrease in the inner columns whilst increase in the outer columns .

Influence of symmetric façade system on the shear force distribution in the outer and inner columns of the structure is plotted in Figure 7-32 and 7-33. From the graph in Figure 7-32, by including symmetric façade system to the structure, a constant 10% increase of the shear forces happens to the structural outer columns. In terms of its

influence to the shear force distribution in the inner columns, the contribution of the symmetric façade system is fluctuating. A major decrease (from 2% to 20%) of the shear forces is shown at most of the storeys (Figure 7-33).

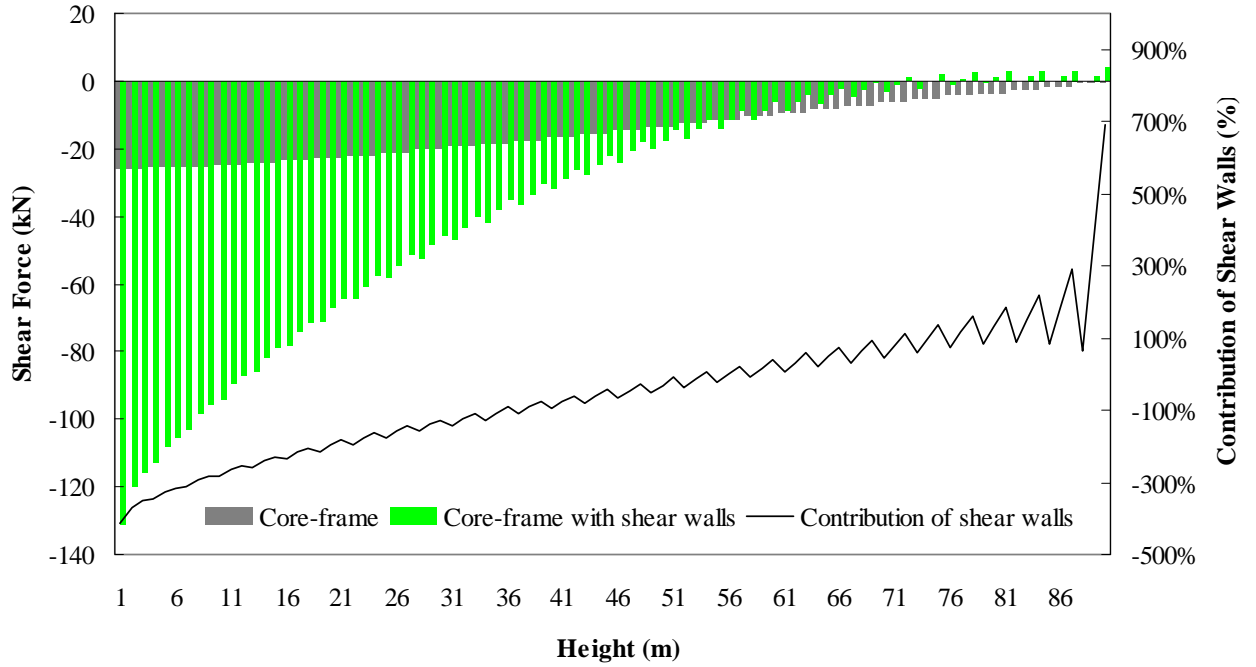


Figure 7-31 Shear force distribution in the inner columns of structural core-frame with and without shear walls

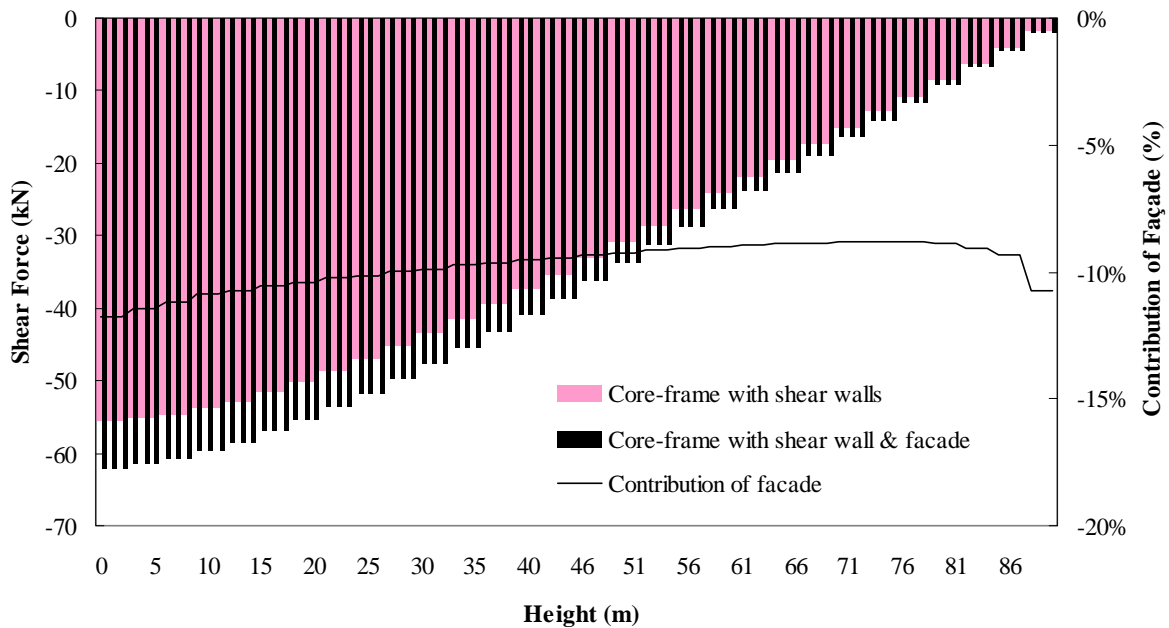


Figure 7-32 Shear force distribution in the outer columns of structural core-frame with and without symmetric shear-wall façade system

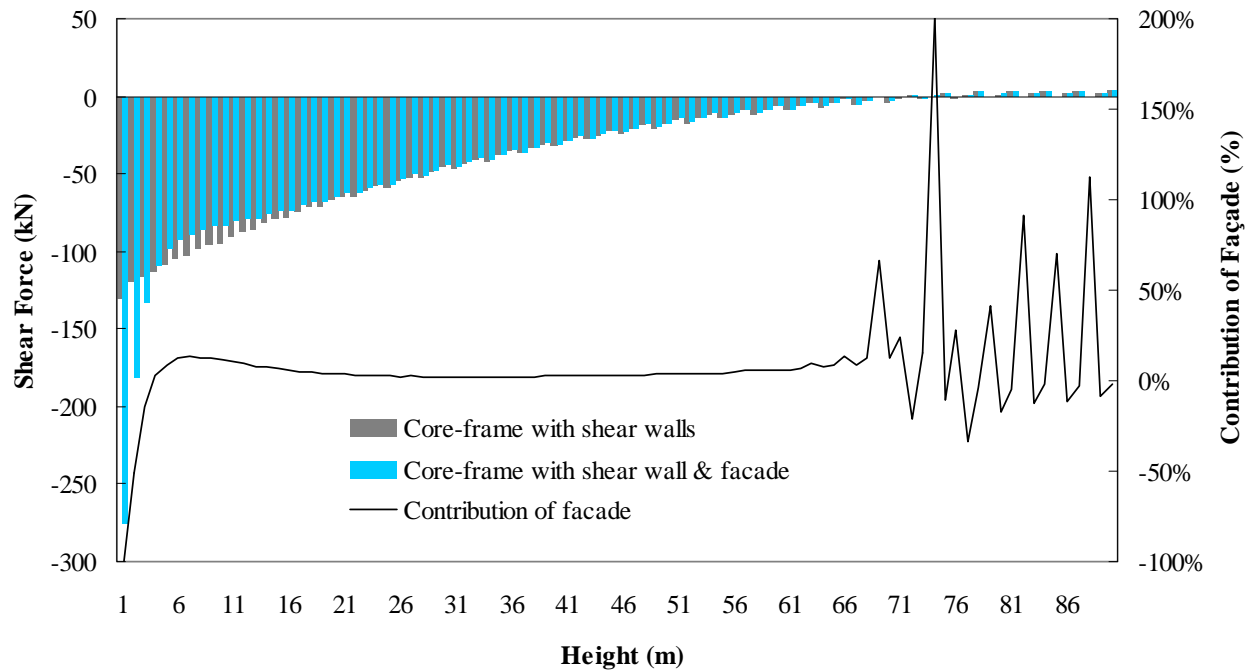


Figure 7-33 Shear force distribution in the inner columns of structural core-frame with and without symmetric shear-wall façade system

7.3.3.3 Influence of Non-structural Components to the Joint Rotation of the Structure

The rotation of the beam/column joints is induced by the deformation of the beam and the column. It has direct relationship with the structure deflection caused by bending moment.

This study analysed the rotational behaviour of structures with different configurations. The contribution of non-structural components was also quantified.

Figure 7-34 plots the rotations of the joints of the structural frame along the inner and outer columns with different non-structural components.

It is clear that the rotations along both inner and outer columns of the structure frame are reduced by including infill walls to the structure. Around 14% decrease of the outer column joint rotation can be achieved by including infill walls to the structural analysis whilst for the inner columns the effect of infill walls to reduce the joint rotation weakens with the increase of the height, with an average of 11% (Figure 7-35).

Figure 7-34 and 7-35 also indicate that the contribution of shear walls in eliminating the frame rotation is significant. More than 30% and 15% constant decrease of the rotations of inner and outer columns respectively can be obtained when adding shear walls to the structure.

Regarding the influence of façade system to the rotational behaviour of the shear wall frame, opposite to the infill walls, the façade system increases the rotation of both inner and outer columns of the structure. By plotting the percentage of the contributions to the frame rotation in Figure 7-35, it can be observed that an average of more than 4% increase of the joint rotations in both inner and outer columns is caused by the inclusion of façade system to the structure.

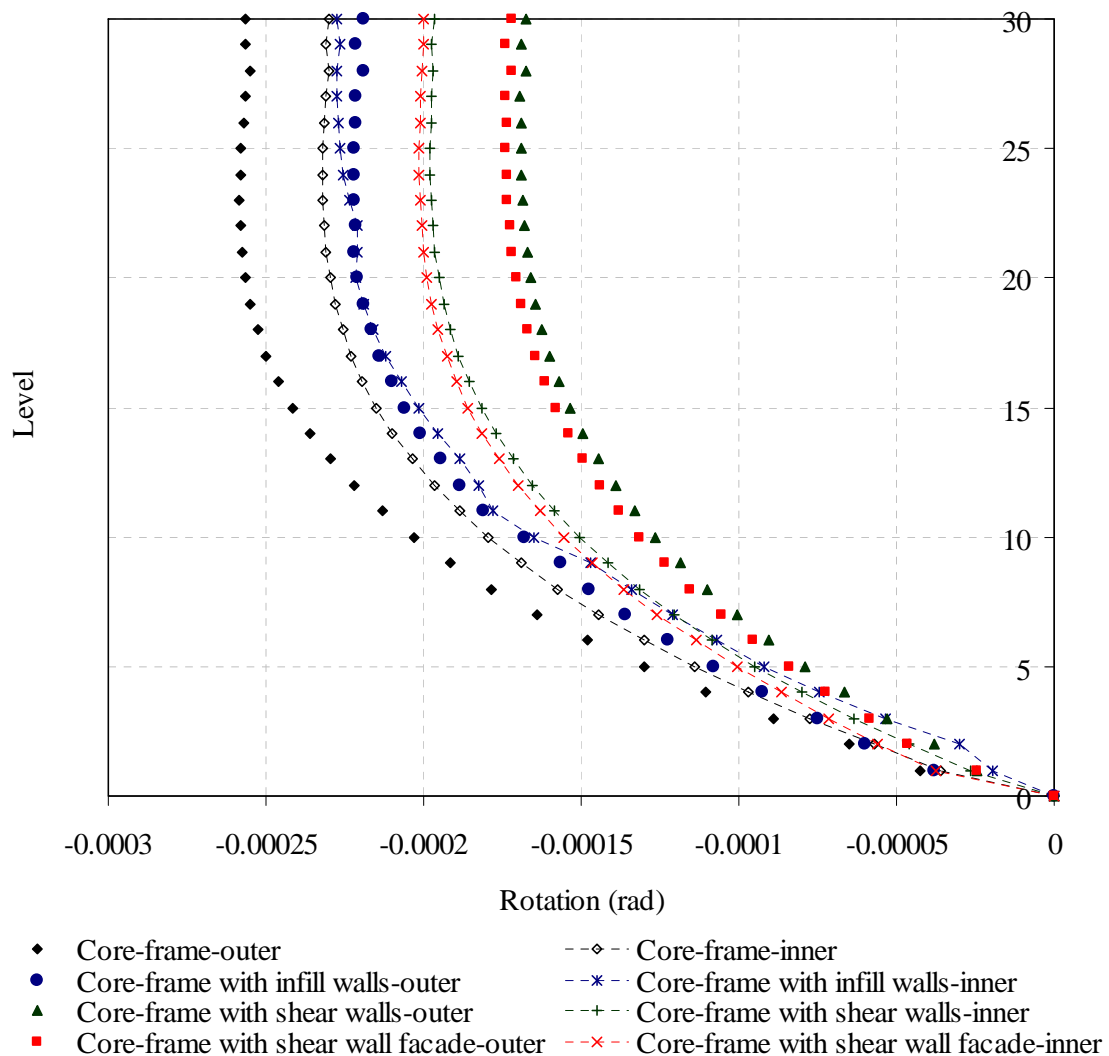


Figure 7-34 Rotational performance of structural frame with different non-structural components

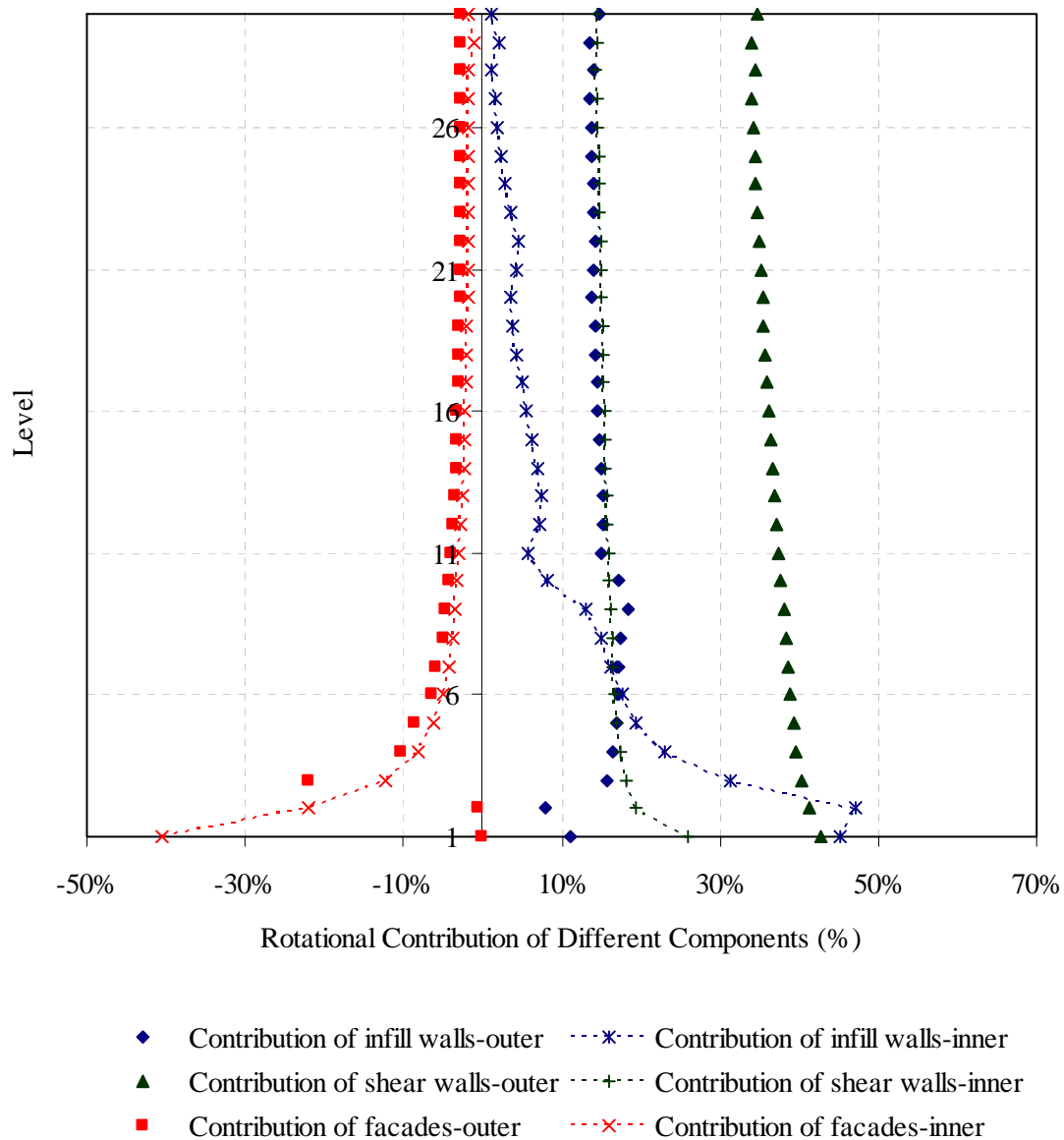


Figure 7-35 The contribution of different non-structural components to the rotation of the structure frame

7.3.4 Conclusions on the Parametric Study

From the parametric study, it can be concluded that by including non-structural components to the structural analysis, discrepancies in the storey drift, natural frequencies of the structure, and the flexural and shear performance of structural elements are identified. The detailed summaries are made as follows:

- The storey drift of tall buildings can be significantly reduced by including infill walls to the structural analysis. Based on this study, approximately 10% of the

lateral deflection can be achieved by adding only two parallel single-bay multi-storey infill walls to the core-frame structure;

- Façade panels can also add stiffness to the tall building structures on condition that the connections between the façade and the structural elements are rigid. Around 10% to 16% stiffness contribution of façade panels is identified in this study;
- From the modal analysis, it is observed that if non-structural components are only included in the parallel direction of the load, there will not be significant contributions of the non-structural components to the fundamental frequency of the structure. However, for the second and third modes, significant changes are identified. If non-structural components are included both parallel and orthogonal to the loading direction, the study shows that more than 7% increase of the fundamental frequency can be achieved by including infill walls in the structural analysis;
- By including infill walls to the structural analysis, the bending moment decreases at most of the storeys in the columns adjacent to the infill walls (to an average of 22% in this study) whilst increases in the other columns which are not adjacent to the infill walls (an average of 17% in this study);
- Shear walls have great influence on the bending moment distributed in the adjacent columns (around 120% decrease in this study) whilst only small changes can be made to the bending moments in the other columns (4% in this study);
- Façade panels do not have significant influence on the bending moment distribution in both inner and outer columns of the structure (less than 2% for all the columns in this study);
- By including infill walls in the analysis, the shear forces in the adjacent columns change significantly (more than 400% increase at the bottom and more than 400% increase at the top of the structure);
- Infill walls increase the shear forces in the outer columns (to a constant 13% in this study);
- Similar to the infill walls, shear walls change the shear force distributions in the adjacent columns significantly (400% increase at the bottom, 600% increase at the

top). And the shear forces in those columns decrease with the height of the structure increases;

- Shear walls decrease the shear forces in the outer columns of the structure (to an average of 4% in this study);
- The inclusion of façade panels to the structural analysis has irregular influence on the shear force distribution in the columns adjacent to the shear walls (decrease at the bottom whilst increase at the top, in this study);
- The inclusion of façade panels to the structural analysis will decrease the shear forces in the outer columns of the structure (to an average of 10% in this study);
- Including infill walls to the tall building structure can reduce the column rotations significantly (to an average of 11% to the adjacent columns and 14% to the other columns);
- Shear walls have dramatic influence on the column rotational behaviour. By including shear walls, the rotation of the adjacent columns can be reduced to an average of 15% and 35% of the rotations in other columns can be achieved in this study;
- Façade panels increase the rotation of the columns (by 5% in this study);

In summary, non-structural components increase the stiffness of the tall buildings. They also change the bending moment and shear force distributions of the structural components, especially those adjacent to the non-structural components. The enhancement of stiffness and change of bending and shear performance of the actual buildings provide the opportunity for refining both the deflection of buildings in the serviceability limit state and the strength design of the structure.

7.4 Conclusions

The analyses conducted in this study detailed the evaluation of the influence from different non-structural components, to diverse aspects of the overall structural performance. From the study, it is concluded that significant influence on the structural stiffness, dynamic performance, flexural and shear strength of the structural elements

can be achieved by including different non-structural components to the structural analysis. In detail, the following conclusions have been drawn.

- At present in Australia, it is a common practice for design engineers to ignore the structural effects of non-structural components in the tall building design. Non-structural components are normally considered as detachments to the structure and isolated from the skeletal structure;
- A noticeable increase in the overall building stiffness occurs by including different non-structural components to the structural analysis;
- The actual drift of real high-rise buildings is usually less than that predicted by the analysis of structural skeleton. The inclusion of non-structural components in part, explains the discrepancy;
- The dominant contribution of non-structural components appears to be the contribution of infill walls. Given that infill walls appear to significantly reduce the storey drift, the contribution of block infill stair wells will also reduce the storey drift;
- In real structures façade panels make a contribution to the stiffness of the overall structure;
- From the modal analysis, it is observed that if non-structural components are only included in the parallel direction of the load, there will not be significant contributions of the non-structural components to the fundamental frequency of the structure. However, for the second and third modes, significant changes are identified. If non-structural components are included both parallel and orthogonal to the loading direction, the study shows that more than 7% increase of the fundamental frequency can be achieved by including infill walls in the structural analysis;
- Given that the non-structural components increase the lateral stiffness, these non-structural components should be further investigated to ensure their integrity and the robustness during the life of the structure;
- Even though the influences of the non-structural components on the lateral performance of the high-rise building are not as apparent as that of the structural

components, for example, shear walls, it is worth paying special attention to the analysis of those non-structural components because of their interaction with the skeletal structure;

In terms of the detailed evaluation results,

- By including infill walls to the structural analysis, the stiffness and the dynamic performance of the structure, as well as the flexural, shear and rotational performance of the structure change dramatically:
 - The storey drift of tall buildings can be significantly reduced by including infill walls to the structural analysis, from 10% (partly infilled frame) to 60% (fully infilled frame) according to the difference of structural configurations;
 - The fundamental frequency of the structure can be greatly changed only when infill walls are installed orthogonal to the loading direction;
 - Nevertheless, when the structure actually reaches the maximum allowable storey drift under the serviceability limit state, owing to the load redistribution caused by the contact between the structural skeleton and the infill wall, the infill wall would have already surpassed its tensile capacity and thus failed;
 - The bending moments decrease at most of the storeys in the columns adjacent to the infill walls whilst increase in the other columns which are not adjacent to the infill walls;
 - The shear forces, however, change significantly in the adjacent columns whilst increase almost constantly in the outer columns;
 - Including infill walls to the tall building structure can reduce the column rotations significantly;
- Shear wall is a structural component. Unsurprisingly, it has significant influence on the overall performance of the structure:
 - More than 27% decrease of the storey drift can be achieved by including shear walls to the structural analysis;
 - Adding shear walls either parallel or perpendicular to the loading direction, the fundamental frequency of the structure increases;

- Shear walls have great influence on the bending moments distributed in the adjacent columns whilst only small changes are made to the bending moments in the other columns;
- Similar to the infill walls, shear walls change the shear force distributions in the adjacent columns significantly (400% increase at the bottom, 600% increase at the top). And the shear forces in those columns decrease with the height of the structure increase;
- Shear walls decrease the shear forces in the outer columns of the structure;
- Shear walls have dramatic influence on the column rotational behaviour. By including shear walls, the rotation of the adjacent columns can be reduced whilst that of the outer columns increase;
- Different from the infill wall, façade panels have irregular influence on the structural performance.
 - The increase of structural stiffness by façade panels mainly depends on the connection/installation conditions of the façade system.
 - Only slight difference can be made to the dynamic performance of the structure by adding façade panels to the structural analysis;
 - In terms of the load bearing capacity of the façade panel, when the building subjects to the maximum allowable storey drift, the analyses showed that the glass panel can well withstand the load redistributed to it;
 - There is no dramatic influence on the bending moment distribution in both inner and outer columns of the structure when taking façade system into consideration;
 - The inclusion of façade panels to the structural analysis reveals façade panels have irregular influence on the shear force distribution in the columns adjacent to the shear walls whilst decrease the shear forces in the outer columns of the structure.

In summary, non-structural components increase the stiffness of the tall buildings. They also have influence on the bending moment and shear force distributions of the structural components, especially those adjacent to them. Moreover, if not isolated

properly, the stress within the non-structural components caused by the load redistribution can lead to the failure of some of the components, for example, the concrete infill wall.

The enhancement of stiffness and change of bending and shear performance of the actual buildings provide the opportunity for refining both the deflection of buildings in the serviceability limit state and the strength design of the structure.

In terms of the design practice, from the analyses in this study, it is recommended that the non-structural components, such as infill walls and façades, should be integrated into the structural analysis by using either the theories provided in this chapter or detailed 3-D finite element models with properly defined connections between the structural and non-structural components. Under this circumstance, the overall performance of a tall building structure can be enhanced and the safety level of the building as well as the individual component can also be improved.

CHAPTER 8

CONCLUSIONS

8.1 Summary and Contributions

The current design approach of tall buildings in Australia requires the structural skeleton to resist vertical and lateral loads, under both ultimate and serviceability loading conditions of the buildings. The non-structural components, such as infill walls, façades and stairs, are treated as non-load bearing components and these components are assumed detached from the primary structure for the design purpose. However, because of different types of physical connections, interactions between the structural skeleton and the non-structural components do occur and both structural and non-structural components participate in resisting structural movements. Various researchers have identified that non-structural components make a considerable contribution to the overall structural performance. Also, according to previous research (Melchers 1990; Hira 2002; Onur et al. 2004), different levels of damage to the non-structural components occur during severe hazards or even in the service life of the structure. These non-structural components often account for the great portion of the total damage to the building under extreme loading events.

Thus, the aim of this study was to analyse the structural performance based on the evaluation of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis. To achieve this specific aim, field reconnaissance, case-study building investigation, laboratory testing and the analysis of tall building structures were conducted, to evaluate the influence of integrating non-structural components into the structural analysis on the overall building performance. The specific non-structural components analysed in this study are infill walls and façades. In depth quantification of the effects caused to the overall structural performance was conducted, followed by design suggestions on the integrated analysis including structural and non-structural components to the structural analysis.

It is discovered that by integrating non-structural components into the structural analysis, building performance differs significantly. From the analyses in this study, when including different non-structural components in the structural analysis, the total

stiffness of the building is significantly increased, to more than 50%, depending on the key influencing factors, which are identified in this study: quantity, location, and connection properties assigned to the non-structural components.

It is also noticed that the natural frequencies of the structure change when different non-structural components are included in the analysis.

In terms of the stress distribution, by including non-structural components in the structural analysis, the bending moment and shear force distributed in the structural components, such as columns, change accordingly. These changes are related to their relevant locations to the specific non-structural components.

The damage level of different non-structural components was also assessed. The maximum allowable structural movements defined by the Australian Standard were applied to the individual non-structural component. It is concluded that if not being delicately isolated from the primary structure, the precast concrete infill panels will not be able to accommodate the amount of stress transferred from the primary structure.

Based on the results obtained from this study, it is concluded that integrating non-structural components into the structural analysis has significant influence on the serviceability of the overall structural system. Damages to the non-structural components caused by the interactions between the primary structure and the non-structural components are also remarkable, even if the whole building system is under service loads. Consequently, the current structural analysis method adopted by the current design practice is suggested to be updated.

8.2 Current Practice

In Chapter 2, findings from literature review are presented. By reviewing the literature, it is observed that even though the significance of including non-structural components to the structural analysis has been widely acknowledged, very limited work has been conducted to consider these secondary elements in the analysis of the overall structural behaviour. An integrated design approach in practice is still lacking.

- Various structural forms and construction materials have been developed with the requirements of taller and stronger buildings. The common forms of the primary structures of tall buildings are rigid frames, frames with shear walls, core frames,

tube in tube structures, braced frames, hybrid structures, etc. Together with the advancement of construction materials, the enhancement of the design of primary structures makes it possible to keep on increasing the height of the world's skyscrapers.

- In current design practice, especially in Australia, only the structural skeleton is analysed and designed for the ultimate limit state and the serviceability limit state according to the design standards. The non-structural components, however, are treated as the detachment to the primary structure as non-load bearing components. Although most of the design standards in different countries require that the non-structural components should be designed as “being isolated from the primary structural system”, it is seldom the case in the construction practice that the primary structure and the non-structural components do not interact. Connections between the structural and non-structural components lead directly to the physical contact between the structural skeleton and non-structural components and hence to interactions.
- There are several key factors that influence the lateral behaviour of tall buildings. They are: the behaviour of the primary structural system, the behaviour of the roof system, the behaviour of the secondary element system, the interactions between the primary and secondary systems and the base-to-height ratio of the building. In terms of the secondary system, the element type, the material properties, location and fixing details subsequently affect the contribution of the secondary elements to the overall structural performance.
- Non-structural components are identified as the components which are attached to or housed in a building or a building systems but are not part of the load resisting system of the building. There are three types of non-structural components: machinery, architectural and electrical components. In this study, architectural components such as infill walls, façades, and stairs are discussed. It is shown from the literature that in modern high-rise structures, the cost of non-structural components can be up to 50% of the total building cost. Moreover, when experiencing earthquakes or high winds, the damage of those non-structural components is the greatest portion of the total monetary loss.
- Some recent research has demonstrated that the actual behaviour of high-rise buildings is very complex because of the conflicting requirements of diverse

(structural and non-structural) building systems. It is identified clearly in previous work (Gad et al. 1998; 1999a; 1999b) that the non-structural components can increase the stiffness and the strength of low-rise structures by more than 100%. However, the actual situation is that some of the non-structural components are likely to be damaged or distressed even under the serviceability loading because of the inadequate design to cope with the possible structural movements.

- The critical factor that integrates the structural skeleton and non-structural components is the connections between the two parts. Broadly, there are three types of connections in a building system: connections between structural elements, connections between non-structural elements and connections between structural and non-structural elements. The connection properties can significantly influence the contribution of each part to the overall building behaviour and thus the overall building performance.
- Up-to-date structural measuring and monitoring technology has the capacity to measure the structural movements in real-time and in the long term. To measure the building movement, especially the movement under service loads, the sensors/measuring equipment must be accurate, reliable, and capable of capturing movements under different frequency ranges. GPS and accelerometers are demonstrated as complementary measuring systems that can be integrated to obtain real-time and long-term data accurately. In various cases GPS is capable of capturing data/ structure movements under low frequencies, whilst accelerometers are suitable for measuring movements at high frequencies.
- Finite element analysis is the most popular analytical tool currently used in both research and industrial practice. It can clearly define the problem and is flexible over a wide range of structures. ANSYS, ABAQUS, SAP, etc. are all popular finite element modelling applications that can efficiently and accurately achieve solutions to structural problems. From the literature, it is also observed that factors such as model simplification approach, element parameters, etc. can significantly influence the final results. Moreover, the computational time and memory are also key considerations when choosing the proper computer simulation software.

8.3 Research Findings

In reviewing the literature, it is identified that even though the importance of non-structural components has been widely recognised, limited work has been conducted to consider this specific topic. Further, the evaluation of the influence of non-structural components to overall structural performance and the various levels of damage inflicted on non-structural components because of the inadequate integration in the design of buildings has not been considered.

Thus, gaps were analysed and the aim and objectives of the study were clearly identified. The aim of this study was to analyse the structural performance based on evaluations of both the global behaviour of buildings and the damage level of individual component by integrating different non-structural components into the structural analysis. In targeting this aim, activities were conducted during the study to meet the specific objectives.

To correspond to the objectives established at the beginning of this study, this section is structured by discussing the findings from this research in relate to the different objectives. In this way, a comprehensive view of the study can be obtained.

Objective 1: Propose and evaluate an integrated analytical system to obtain reliable data from building movements and to process the analysis

In this study, field investigation and laboratory testing were conducted to demonstrate the building performance, facilitated by the finite element analyses and theoretical validation. The holistic approach of including all the above has been demonstrated as a sophisticated and reliable system because of the mutual validation process and the correlating results obtained from each step.

The design and integration of the measuring system is one of the critical parts in the laboratory testing in this study. Thus, detailed sensor selection and calibration were conducted and discussed during the preparation of the preliminary design of the measurement system (Li et al. 2007). Some findings were obtained from the sensor calibration.

- Because of obvious advantages such as low self-weight, low cost and relatively high-accuracy, a group of Micro-electro-mechanical-systems (MEMS) sensors were investigated. Both static and vibration tests were conducted for the verification of the reliability, repeatability, and accuracy of these sensors. However, from the

testing results, it is noted that even though all the tests of the MEMS sensors are repeatable, the reliability and measuring accuracy are inconsistent because of the limitation of their measuring capacities. Moreover, constrained by cables and external power sources, the MEMS sensors tested are concluded to be not suitable for this specific research.

- However, the Dytran accelerometers, which were tested together with the MEMS sensors, show high accuracy, repeatability, and reliability, especially when at the frequency greater than 2Hz. Thus, Dytran series accelerometers were chosen for the measuring system in this research.
- GPS is proved not suitable for the tests in this study. Since a scaled laboratory model was involved in the test, considering the relative mass and the high requirement on the weather conditions, as well as the sensitivity limit, even though the advantages of integrating GPS into the traditional structural monitoring system are immense, it is decided that only traditional structural measuring system should be involved and if possible, GPS component would be integrated into the measurement system in the real building test in the future.

In terms of the analytical method involved in this study, identical results obtained from the theoretical analyses and the parametric study prove that the combination of these two approaches is reliable and effective in evaluating the influence of different non-structural components to the overall structure performance.

Objective 2: Identify the effects of integrating non-structural components into the structural analysis on the overall building performance

Objective 3: Identify the influence of connection properties to the overall structural performance

These two objectives are discussed together based on the coherent conclusions drawn from both the laboratory testing and the finite element analyses on the basis of the information collected from the field reconnaissance. Significant contributions are made by including non-structural components to the structural analyses. Being specific, following findings are observed.

Findings from the laboratory tests

In Chapter 6, details of the model design and laboratory testing are presented. A 1:100-scaled model was designed and tested in the laboratory, with different structural configurations and connection properties.

From the results, it is noticed that the cable arrangement can affect the final results a lot, whilst the bottom fixing details only has minor influence on the data obtained from the tests.

It is also identified from the results that the stiffness of the structure frame increases significantly after attaching infill panels to it.

- When the infill panels are rigidly connected to the frame, apparently they work almost like walls. The lateral stiffness of the structure thus dramatically increases (more than 95%).
- If changing the connection details between the infill panels and the frame, the lateral stiffness of the system changes accordingly. The pin-connected infill panels can also bring significant enhancement (more than 50%) to the stiffness of the structural frame.

For the purpose of further prediction and simulation, finite element models were developed and calibrated based on the laboratory testing. From Chapter 6, the maximum difference between the finite element and experimental data is less than 30%, which is deemed satisfactory.

It is concluded from the experiment that the stiffness of a tall building structure can be significantly increased by including non-structural components to the structural analysis. The amount of stiffness increase is also influenced by the connection properties between the non-structural components and the primary structure.

It is also noticed that the finite element models and the modelling techniques developed according to the testing results in this study can be used in the future analysis with a high confidence. In the analyses, the finite element model was calibrated by the laboratory testing results. To validate the reliability of the model and the conclusions drawn from the transient analyses, a series of static analyses were also conducted. The results appeared to be highly consistent and reasonable.

Findings from the laboratory testing validated the preliminary conclusions drawn from the case study building analysis. It demonstrates that the non-structural components can bring significant amount of extra stiffness to the primary structure, if being included into the structural analysis. Considering that in the current practice, these non-structural components are all treated as components detached to the primary structure and are not taken into account in the design process, the conclusions drawn from this study reveal potential benefits of having these components included in the structural analysis.

However, findings in the experimental program are based on the testing and analysis implemented on the laboratory model, some of the performance such as the natural frequency and the stress distribution within different non-structural components can not be fully evaluated through the test. Under this circumstance, more detailed analyses were carried out to the full-scaled model in order that the findings in this study can be consolidated.

Findings from the analyses of a case-study building and the tall building structures

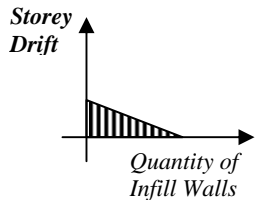
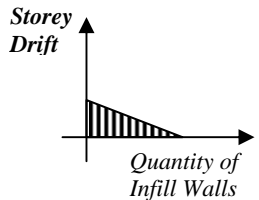
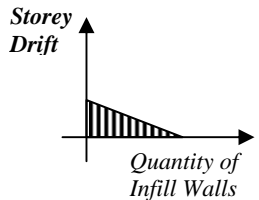
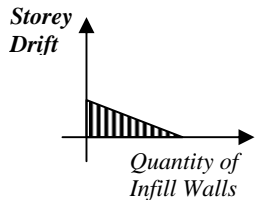
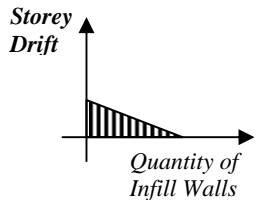
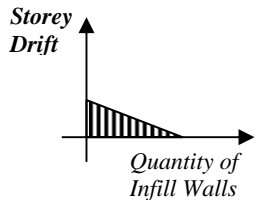
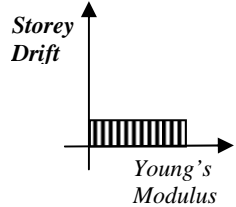
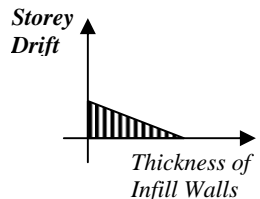
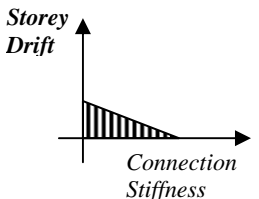
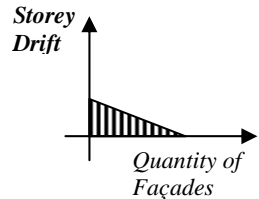
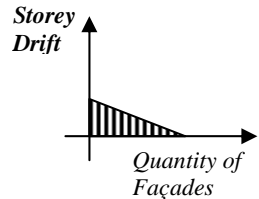
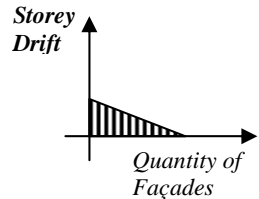
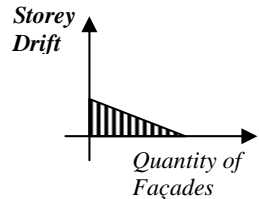
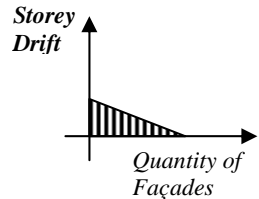
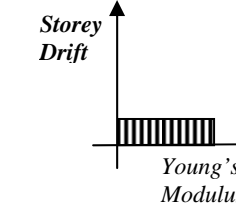
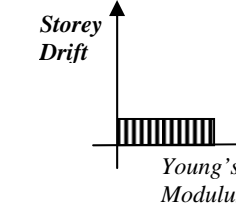
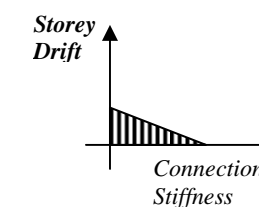
In Chapter 5, detailed analyses based on a case-study building were conducted, focusing on the evaluation of the sensitive parameters, such as the location, quantity and rigidity of connections of the non-structural components, which may influence the contribution of the non-structural components to the structural performance. On the basis of the conclusions drawn from the case-study building analysis and the laboratory testing, a detailed finite element analysis and theoretical calculations were undertaken to predict and quantify the influence of integrating non-structural components into the overall structural analysis (Chapter 7). A typical building model was developed, with steel frame and concrete shear cores as its primary structural system. Infill walls and façades were included at different stages to represent different types of non-structural components. From the results, the following conclusions can be drawn.

- Integrating non-structural components into the overall structural analysis has significant influence on the overall structural performance.
 - ***The influence on the storey drifts of the building.*** In Table 8-1, some influencing factors are listed, with estimation of their influence. Generally, infill walls have a bracing effect on the structural frame, thus extra stiffness can be achieved by including infill walls to the structural analysis. By treating a

façade system as an outrigger system, the contribution of façade to the structural stiffness can be evaluated.

- ***The influence on the natural frequency of the building.*** From the modal analysis, it is observed that if non-structural components are only included in the parallel direction of the load, there is no significant contribution of the non-structural components to the fundamental frequency of the structure. However, for the second and third modes, significant changes can be identified. If non-structural components are included both parallel and orthogonal to the loading direction, the study shows that more than 7% increase of the fundamental frequency can be achieved by including infill walls in the structural analysis (detailed results refer to Chapter 7).
- Integrating non-structural components into the overall structural analysis has a dramatic influence on the load distribution in the primary structural elements.
 - ***Influence on the bending moment distributions in columns.*** From this study, it is identified that infill walls can decrease the bending moment distributed in adjacent columns to approximately 20% whilst attracting about 18% extra bending moment to the outside columns. Façade panels influence the bending moment distribution in both adjacent and outside columns in a more subtle and varying way: 2% and 4%, respectively.
 - ***Influence on the shear force distributions in columns.*** In contrast to the influence on the bending moment distribution, including infill walls in the analysis, the shear force distributed in the adjacent columns increases at the bottom of the building whilst decreases at the top: from more than 400% (increase of the absolute value) to less than -400% (decrease of the absolute value). For the outside columns, an almost constant increase of more than 13% in the shear force can be observed. Similarly, façade systems bring a constant increase of the shear force to the outside columns whilst a varying decrease can be identified in the adjacent columns.

Table 8-1 Influence on the storey drift by integrating non-structural components into the structural analysis

Influence on the Storey Drift of the Building								
	Quantity		Location		Property		Connection	
	None	Full	Along Loading Direction	Across Loading Direction	Material Property	Dimension (Increase Thickness)	Flexible	Rigid
Infill Wall	Storey drift decreases significantly with the increase of the number of infill walls	Storey drift decreases significantly if install infill walls along the loading direction	Install infill walls along the loading direction will not have big Influence on the storey drift	Change of material properties of infill walls won't change too much of the storey drift	Increase the thickness of infill walls decreases the storey drift	Storey drift decreases with the increase of the connection stiffness		
								
Façade	Storey drift decreases with the increase of the number of façade	Storey drift decreases if install façade along the loading direction	Install façade across the loading direction will not have big Influence on the storey drift	Change of material properties of façade won't change too much of the storey drift	Increase the thickness of façade won't change too much of the storey drift	Storey drift decreases significantly with the increase of the connection stiffness		
								

Objective 4: Identify the damage level of individual non-structural components when integrating non-structural components into the structural analysis

In Chapter 7, detailed discussions on the stress distribution within different non-structural components are provided. It is concluded that when the building is under the maximum allowable serviceability movements set by the Australian Standards, the infill wall is hard to accommodate the deformation caused by the storey drift given that it is not deliberately isolated from the structural frame (as discussed in Chapter 7). Thus, unexpected damage will happen to the infill walls. However, the glass panels can work well under the limit of the in-plane movement/deformation caused by the storey drift of the structure.

Objective 5: Propose integrated design suggestions for structural and non-structural components of multi-storey buildings

In summary, it is clear that the integration of non-structural components into the structural analysis has significant effects on the overall structural performance. Consequently, the design of structural elements will be influenced because of the changes of bending moment and shear force distributions. Moreover, the damage always occurs to the non-structural components even under the serviceability load because of the interactions between the structure and the non-structural components. Hence, it is highly desirable that the integration of non-structural components into the overall structural analysis being implemented in the design practice.

8.4 Recommended Future Work

This study investigated certain types of non-structural components. A scaled model was tested, followed by the finite element analyses. In terms of the future work, there are several recommendations:

- Further investigation should be carried out to other types of non-structural components, especially those built-in non-structural components in the building systems, such as stairs, etc.;
- The full-scale testing is recommended in the future research. Full-scale, real-time, long-term structural monitoring will enhance the accuracy in quantifying the contributions of non-structural components to the overall building performance.

However, the identification and control of variables are the key issues need to be addressed;

- It is suggested that the measuring system should be updated. Since the full-scale, real-time, long-term testing is recommended, the integration of GPS into the measuring system is necessary and will be beneficial, as discussed during the literature review of this thesis;
- Although it is believed that this study reveals potential benefits to the building industry by integrating non-structural components into the structural analysis, subsequent issues such as the extra cost brought by the additional modelling and the uncertain properties of non-structural components, etc. may be of greater concern of the industry. Thus, based on the results obtained from this study, simplified models or broader involvement of industry friendly modelling software (such as ETABS) can be considered in the future work;
- International collaborations and investigations are recommended in order that the contents and applications of the study can be globalised;
- Further investigation on the design approach stated in this study is also suggested, aiming to propose detailed recommendations/ guidelines for relevant design standards both in Australia and internationally.

REFERENCES

- (2002). "Appartementhuser = Apartment Buildings." *K. Krmer*, Stuttgart.
- Ahmad, S. and Kumar, K. (2001). "Interference Effects on Wind Loads on Low-rise Hip Roof Buildings." *Engineering Structures*, Vol. 23, No. 12, pp 1577-1589.
- Ahmed, B. and Nethercot, D. A. (1998). "Effect of Column Axial Load on Composite Connection Behaviour." *Engineering Structures*, Vol. 20, No. 1-2, pp 113-128.
- Ajrab, J. J., Pekcan, G., and Mander, J. B. (2004). "Rocking Wall--Frame Structures with Supplemental Tendon Systems." *Journal of Structural Engineering*, Vol. 130, No. 6, pp 895-903.
- Allen, H. V., Terry, S. C. and De Bruin, Diederik W. (1989). "Accelerometer Systems with Self-testable Features." *Sensors and Actuators: A Special Issue Devoted to Micromechanics*, Vol. 20, No. 1-2, pp 153-161.
- Aoyama, H. (2001). "Design of Modern Highrise Reinforced Concrete Structures." *Imperial College Pres*, London.
- Arnold, C. (1991). "The Seismic Response of Non-structural Elements in Buildings." *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 24, No. 4, pp 306-316.
- Arnold, C., Hopkins, D., and Elessor, E. (1987). "Design and Detailing of Architectural Elements for Seismic Damage Control." *Building Systems Development Inc., KRTA Ltd. And Forell/Elessor Engineering Inc.*
- Arup. (2008). "The Structural Design and Calculation Book of Jiali Center Office Building." *Arup Beijing Office*, Internal, Beijing, China.
- Australian Bureau of Statistics (2000). "Yearbook of Australia, 2000." *Australian Bureau of Statistics*, Canberra, Australia.
- Aziz, W., Othman Z., W. A., and H, N. (2006). "Monitoring High-rise Building Deformation Using Global Positioning System." *GIS Development*, <<http://www.gisdevelopment.net/technology/gps/techgp0024.htm>> (March 2006).

Bayo, E., Cabrero, J. M. and Gil, B. (2006). "An Effective Component-based Method to Model Semi-rigid Connections for The Global Analysis of Steel and Composite Structures." *Engineering Structures*, Vol. 28, No. 1, pp 97-108.

Beedle, L. S. and Rice, D. B. (1996). "Tall Building Structures--A World View : Proceedings of 67th Regional Conference in Conjunction with ASCE Structures Congress XIV, Chicago, Illinois, USA, April 15-18, 1996." *Bethlehem, Pa.*, Council on Tall Buildings and Urban Habitat Lehigh University.

Bhat, R. B. and Chakraverty, S. (2004). "Numerical Analysis in Engineering." *Alpha Science*, Pangbourne.

Binder, G. (2001). "Tall Buildings of Asia and Australia" *Images Publishing Group*, Mulgrave, Victoria, Australia.

Blake, L. S. (1989). "Civil Engineer's Reference Book (Fourth Edition)." *Reed Educational and Professional Publishing Ltd.*, Oxford, UK.

Bonato, P., Ceravolo, R., De Stefano, A. and Molinari, F. (2000). "Cross-time Frequency Techniques for the Identification of Masonry Buildings." *Mechanical Systems and Signal Processing*, Vol. 14, No.1, pp 91-109.

Bonato, P., Ceravolo, R., De Stefano, A. and Knaflitz, M. et al. (1997). "Bilinear Time-Frequency Transformations in the Analysis of Damaged Structures." *Mechanical Systems and Signal Processing*, Vol. 11, No. 4, pp 509-527.

British Standards (1998). "Eurocode 8: Design Provisions for Earthquake Resistance of Structures." *BSI British Standards*, UK.

Brockenbrough, R. L. and Merritt, F. S. 1999. "Structural Steel Designer's Handbook." *McGraw-Hill*, New York.

Brüel and Kjar (1974). "Accelerometers : Instructions and Applications." *Narum*, Denmark.

Brunsdon, D., and Clark, W. (2001). "Modern Multi-storey Buildings and Moderate Earthquakes." *Proceedings of the Annual Technical Conference of the New Zealand Society for Earthquake Engineering*, New Zealand.

Cabrero, J. M. and E. Bayo (2005). "Development of Practical Design Methods for Steel Structures with Semi-rigid Connections." *Engineering Structures*, Vol. 27, No. 8, pp 1125-1137.

Calado, L. and Lamas, A. (1998). "Seismic Modelling and Behaviour of Steel Beam-to-column Connections." *Journal of Constructional Steel Research, Second World Conference on Steel in Construction*, Vol. 46, No. 1-3, pp 258-259.

Casafont, M., Arnedo, A. Roure, F. and Rodriguez-Ferran, A. (2006). "Experimental Testing of Joints for Seismic Design of Lightweight Structures. Part 2: Bolted Joints in Straps." *Thin-Walled Structures*, Vol. 44, No. 6, pp 677-691.

Celebi, M. (2000). "GPS in Dynamic Monitoring of Long-period Structures." *Soil Dynamics and Earthquake Engineering*, Vol. 20, No. 5-8, pp 477-483.

Centre, N. R. (1994). "Assessment of Damage to Residential Buildings Caused by Northridge Earthquake.", *Prepared for U.S. Department of Housing and Urban Development*, Washington, D.C..

Chan, W. S., Xu, Y. L., Ding, X. L. and Dai, W. J. (2006). "An Integrated GPS-accelerometer Data Processing Technique for Structural Deformation Monitoring." *Journal of Geodesy*, Vol. 80, No. 12, pp 705-719.

Chen, C. C., Lin, C. C. and Tsai, C. L. (2004). "Evaluation of Reinforced Connections between Steel Beams and Box Columns." *Engineering Structures*, Vol. 26, No. 13, pp 1889-1904.

Chen, S. and Du, G. (2007). "Influence of Initial Imperfection on the Behaviour of Extended Bolted End-plate Connections for Portal Frames." *Journal of Constructional Steel Research*, Vol. 63, No. 2, pp 211-220.

Cheng, C. T. and Chen, C. C. (2005). "Seismic Behavior of Steel Beam and Reinforced Concrete Column Connections." *Journal of Constructional Steel Research*, Vol. 61, No. 5, pp 587-606.

Cohen, M. P. (1986). "Design Solutions Utilizing the Staggered-steel Truss System." *AISC National Engineering Conference*, Nashville, TN.

Council on Tall Buildings and Urban Habitat. (2001). "Tall Buildings and Urban Habitat: Cities in the Third Millennium." *Council on Tall Buildings and Urban Habitat, Spon Press*, New York.

de Lima, L. R. O., de Andrade, S. A. L., da S. Vellasco, P. C. G. and da Silva, L. S. (2002). "Experimental and Mechanical Model for Predicting the Behaviour of Minor Axis Beam-to-column Semi-rigid Joints." *International Journal of Mechanical Sciences*, Vol. 44, No. 6, pp 1047-1065.

Dean, B., Martin, O., Emery, D., and Chancellor, P. (2001). "Tall Building Design Innovations in Australia." *6th World Congress of the Council On Tall Buildings and Urban Habitat, Spon Press*, New York.

Dedolph Jr., C. T. and Jaselskis, E. J. (1997). "Innovative Wall System for Construction Industry." *Journal of Architectural Engineering*, Vol. 3, No. 4, pp 176-184.

Deierlein, G. G. (1997). "Steel-framed Structures." *Progress in Structural Engineering and Materials*, Vol. 1, No. 1, pp 10-17.

Dissanayake, U. I., Davison, J. B. and Burgess, I. W. (1999). "Composite Beam Behaviour in Braced Frames." *Journal of Constructional Steel Research*, Vol. 49, No. 3, pp 271-289.

Doherty, K., Griffith, M. C., Lam, N. and Wilson, J. (2002). "Displacement-based Seismic Analysis for Out-of-plane Bending of Unreinforced Masonry Walls." *Earthquake Engineering and Structural Dynamics*, Vol. 31, pp 833-850.

Eisele, J. and Kloft, E. (2003). "High-rise Manual: Typology and Design, Construction and Technology." *Birkhauser-Publishers for Architecture*, Boston, MA.

Ekwueme, C. G., Srinivasan, M. and Hart, G. C. (1997). "Effect of Flanged Walls on the Seismic Performance of Tall Buildings." *The Structural Design of Tall Buildings*, Vol. 6, No. 4, pp 263-277.

El-Rabbany, A. (2006). "Introduction to GPS : the Global Positioning System." *Artech House*, Boston, MA.

Fan, H., Li, Q.S., Tuan, A. Y. and Xu L. (2009) "Seismic Analysis of the World's Tallest Building" *Journal of Constructional Steel Research*, Vol. 65, No. 5, pp 1206_1215.

Foutch, D. A. and Yun, S. Y. (2002). "Modeling of Steel Moment Frames for Seismic Loads." *Journal of Constructional Steel Research*, Vol. 58, No. 5-8, pp 529-564.

Freeman, S. A. (1977). "Racking Tests of High-rise Building Partitions." *Journal of the Structural Division, ASCE*, Vol.103, No. 8, pp 1673-1685.

Fu, F. and Lam, D. (2006). "Experimental Study on Semi-rigid Composite Joints with Steel Beams and Precast Hollowcore Slabs." *Journal of Constructional Steel Research*, Vol. 62, No. 8, pp 771-782.

Fu, J. Y., Li, Q. S. and Xie, Z. N. (2006). "Prediction of Wind Loads on a Large Flat Roof using Fuzzy Neural Networks." *Engineering Structures*, Vol. 28, No. 1, pp 153-161.

Fujii, Y. (2007). "Impact Response Measurement of an Accelerometer." *Mechanical Systems and Signal Processing*, Vol. 21, No. 5, pp 2072-2079.

Gad, E. F. and Duffield, C. F. (2000). "Lateral Behaviour of Light Framed Walls in Domestic Structures." *Proceedings of the 12th World Conference on Earthquake Engineering*, Auckland, NZ.

Gad, E. F., C. F. Duffield, Chandler, A. M., and Stark, G. (1998). "Testing of Cold-formed Steel Framed Domestic Structures." *Proceedings of the 11th European Conference on Earthquake Engineering*, France.

Gad, E. F., Chandler, A. M., Duffield, C. F. and Stark, G. (1999). "Lateral Behavior of Plasterboard-Clad Residential Steel Frames." *Journal of Structural Engineering*, Vol. 125, No. 1, pp 32-39.

Gad, E. F., Duffield, C. F., Hutchinson, G. L., Mansell, D. S. and Stark, G. (1999). "Lateral Performance of Cold-formed Steel-framed Domestic Structures." *Engineering Structures*, Vol. 21, No. 1, pp 83-95.

Gad, E. F., Duffield, C. F., Chandler, Adrian M. and Stark, G. (1998). "Testing of Cold-formed Steel Framed Domestic Structures." *the 11th European Conference on Earthquake Engineering*, France.

Gizejowski, M. A., Papangelis, J. P. and Parameswar, H. C. (1998). "Stability Design of Semi-continuous Steel Frame Structures." *Journal of Constructional Steel Research*, *Second World Conference on Steel in Construction*, Vol. 46, No. 1-3, pp 99-101.

Global Seismic Hazard Assessment Program. (1992). "Global Seismic Hazard Map." <<http://www.seismo.ethz.ch/GSHAP/>> (20 Jan, 2009).

Grierson, D. E. and Khajehpour, S. (2002). "Method for Conceptual Design Applied to Office Buildings." *Journal of Computing in Civil Engineering*, Vol. 16, No. 2, pp 83-103.

Griffith, M. C. and Alaia, R. (1997). "Gap Effects on the Seismic Ductility of Brick Infilled Concrete Frames." *Proceedings of The Mechanics of Structures and Materials*, Balkema, Rotterdam.

Hall, J. F. (Technical Editor) (1995). "Nonstructural Damage, Northridge Earthquake of January 17, 1994 Reconnaissance Report." *Earthquake Spectra*, Vol. 1, pp 453-513.

He, J. and Song, C. C. S. (1997). "A Numerical Study of Wind Flow around the TTU Building and the Roof Corner Vortex." *Journal of Wind Engineering and Industrial Aerodynamics*, *Computational Wind Engineering*, No. 67-68, pp 547-558.

Hira, A., Duffield, C. F., Gad, E. F. (2002). "Seismic Design Requirements for Non-structural Components of High-rise Buildings." *Master Thesis*, The University of Melbourne.

Hoenderkamp, J. C. D. (2004). "Shear Wall with Outrigger Trusses on Wall and Column Foundations." *The Structural Design of Tall and Special Buildings*, Vol. 13, No. 1, pp 73-87.

Hoenderkamp, J. C. D. and Snijder, H. H. (2003). "Preliminary Analysis of High-Rise Braced Frames with Façade Riggers." *Journal of Structural Engineering*, Vol. 129, No. 5, pp 640-647.

Horr, A. M., Safi, M. (2003). "Multi-layered Energy Absorber Frames for Tall Buildings under High-speed Impact." *The Structural Design of Tall and Special Buildings*, Vol. 12 No. 5, pp 423-450.

Gunel, H. M. and Ilgin, E. H. (2007). "A Proposal for the Classification of Structural Systems of Tall Buildings" *Building and Environment*, Vol. 42, No. 7, pp 2667–2675.

Hidalgo, P. A., Jordan, R. M. and Martinez, M. P. (2002). "An Analytical Model to Predict the Inelastic Seismic Behavior of Shear-wall, Reinforced Concrete Structures." *Engineering Structures*, Vol. 24, No. 1, pp 85-98.

Jan, T. S., Liu, M. W. and Kao, Y. C. (2004). "An Upper-bound Pushover Analysis Procedure for Estimating the Seismic Demands of High-rise Buildings." *Engineering Structures*, Vol. 26, No. 1, pp 117-128.

Kabche, J. P., Caccese, V., Berube, K. A. and Bragg, R. (2007). "Experimental Characterization of Hybrid Composite-to-metal Bolted Joints under Flexural Loading." *Composites Part B: Engineering*, Vol. 38, No. 1, pp 66-78.

Katagiri, J., Ohkuma, T. and Marukawa, H. (2002). "Analytical Method for Coupled Across-wind and Torsional Wind Responses with Motion-induced Wind Forces." *Journal of Wind Engineering and Industrial Aerodynamics*, Vol. 90, No. 12-15, pp 1795-1805.

Kemp, A. R. and Nethercot, D. A. (2001). "Required and Available Rotations in Continuous Composite Beams with Semi-rigid Connections." *Journal of Constructional Steel Research*, Vol. 57, No.4, pp 375-400.

Kicinger, R., Arciszewski, T. and DeJong, K. (2005). "Evolutionary Design of Steel Structures in Tall Buildings." *Journal of Computing in Civil Engineering*, Vol. 19, No. 3, pp 223-238.

Kim, H. S. and Lee, D. G. (2003). "Analysis of Shear Wall with Openings using Super Elements." *Engineering Structures*, Vol. 25, No. 8, pp 981-991.

Kim, H. S., Lee, D. G. and Kim, C. K. (2005). "Efficient Three-dimensional Seismic Analysis of a High-rise Building Structure with Shear Walls." *Engineering Structures*, Vol. 27, No. 6, pp 963-976.

Kirke, A. and Hao, H. (2004). "Estimation of Failure Probabilities of RC Frame Structures in Singapore to the Simulated Largest Credible Ground Motion." *Engineering Structures*, Vol. 26, No. 1, pp 139-150.

Kuang, J. S. and Li, S. (2005). "Interaction-Based Design Formulas for Transfer Beams: Box Foundation Analogy." *Practice Periodical on Structural Design and Construction*, Vol. 10, No. 2, pp 127-132.

Kudzys, A. (2006). "The Structural Safety of Connections with Fillet Welds." *Journal of Constructional Steel Research*, Vol. 62, No. 12, pp 1238-1243.

Langdon, G. S. and Schleyer, G. K. (2004). "Modelling the Response of Semi-rigid Supports under Combined Loading." *Engineering Structures*, Vol. 26, No. 4, pp 511-517.

Lee, D.-G., Kim, H. S. and Hah Chun, M. (2002). "Efficient Seismic Analysis of High-rise Building Structures with the Effects of Floor Slabs." *Engineering Structures*, Vol. 24, No. 5, pp 613-623.

Letchford, C. W., Denoon, R. O. Johnson, G. and Mallam, A. (2002). "Dynamic Characteristics of Cantilever Grandstand Roofs." *Engineering Structures*, Vol. 24, No. 8, pp 1085-1090.

Li, B. Kealy, A., Duffield, C. F. and Hutchinson, G. L. (2007). "Evaluating the Performance of Low-Cost Inertial Sensors for use in Integrated Positioning Systems." *Proceedings of IGNSS2007*, Sydney, Australia.

Li, Q. S. (2001). "Stability of Tall Buildings with Shear-wall Structures." *Engineering Structures*, Vol. 23, No. 9, pp 1177-1185.

Liew, J. Y. R., T. H. Teo, Shanmugam, N. E. and Yu, C. H. (2000). "Testing of Steel-concrete Composite Connections and Appraisal of Results." *Journal of Constructional Steel Research*, Vol. 56, No. 2, pp 117-150.

Liew, Y. L., Duffield, C. F. and Gad, E. F. (2002). "The Influence of Plasterboard Clad Walls on the Structural Behaviour of Low Rise Residential Buildings." *Electronic Journal of Structural Engineering*, Vol. 1, No. 1, pp1-16.

Lim, J. B. P. and Nethercot, D. A. (2004). "Stiffness Prediction for Bolted Moment-connections between Cold-formed Steel Members." *Journal of Constructional Steel Research*, Vol. 60, No. 1, pp 85-107.

Lu, X. and Chen, Y. (2005). "Modeling of Coupled Shear Walls and Its Experimental Verification." *Journal of Structural Engineering*, Vol. 131, No. 1, pp 75-84.

Lu, X., Zou, Y., Lu, W. and Zhao, B. (2007). "Shaking table Model Test on Shanghai World Financial Center Tower." *Earthquake Engineering and Structural Dynamics*, Vol. 36, No. 4, pp 439-457.

Lu, Z. R. and Law, S. S. (2006). "Identification of Prestress Force from Measured Structural Responses." *Mechanical Systems and Signal Processing*, Vol. 20, No. 8, pp 2186-2199.

MacGregor, J. G. (1997). "Reinforced Concrete : Mechanics and Design." *Prentice Hall*, Upper Saddle River, N.J..

MacKerras, C. and Yorke, A. (1991). "The Cambridge Handbook of Contemporary China." *Cambridge University Press*, Cambridge, England, UK.

Mahendran, M. and Moor, C. (1999). "Three-Dimensional Modeling of Steel Portal Frame Buildings." *Journal of Structural Engineering*, Vol. 125, No. 8, pp 870-878.

Mark, W. D. and Reagor, C. P. (2007). "Static-transmission-error Vibratory-excitation Contributions from Plastically Deformed Gear Teeth Caused by Tooth Bending-fatigue Damage." *Mechanical Systems and Signal Processing*, Vol. 21, No. 2, pp 885-905.

Masarira, A. (2002). "The Effect of Joints on the Stability Behaviour of Steel Frame Beams." *Journal of Constructional Steel Research*, Vol. 58, No. 10, pp 1375-1390.

Massey, W. and Megget, L. (1992). "Architectural Design for Earthquake: a Guide to the Design of Non-structural Elements." *Krta Limited*.

McDonnell, B. (2001). "What Businesses Learned from the Nisqually Earthquake of Febraury 28, 2001." *The Cascadia Region Earthquake Workgroup (CREW)*, Seattle (www.crew.org).

McGorry, R. W., DiDomenico, A. Chang, C. C. (2007). "The Use of a Heel-mounted Accelerometer as an Adjunct Measure of Slip Distance." *Applied Ergonomics*, Vol. 38, No. 3, pp 369-376.

Melchers, R. E. (Editor). (1990). "Newcastle Earthquake Study." *The Institution of Engineers*, Australia.

Moaveni, S. (1999). "Finite Element Analysis : Theory and Application with ANSYS." *Prentice Hall*, Upper Saddle River, N.J..

Mondal, G. and Jain, S. K. (2005). "Design of Non-structural Elements for Buildings: a Review of Codal Provisions." *The Indian Concrete Journal*, Vol. 79; No. 8, pp 22-32.

Monto, G., Nuti, C., and Santini, S. (1996). "Seismic Assessment of Hospital Systems." *Proceedings of the 11th WCEE*, Acapulco, Mexico.

Naeim, F. (1999). "Lessons Learned from Performance of Non-structural Components during the January 17, 1994 Northridge Earthquake - Case Studies of Six Instrumented Multistory Buildings." *Journal of Seismology and Earthquake Engineering*, Vol. 2, No. 1, pp 47-57.

Naeim, F., (2001). "The Seismic Design Handbook." *Kluwer Academic Publishers*, Boston, London.

Nawy, E. G. (2003). "Reinforced Concrete: a Fundamental Approach." *Prentice Hall*, Upper Saddle River, N.J..

Nethercot, D. A. (2000). "Frame Structures: Global Performance, Static and Stability Behaviour: General Report." *Journal of Constructional Steel Research*, Vol. 55, No. 1-3, pp 109-124.

Neves, L. A. C., Cruz, P. J. S. and Henriques, A. A. R. (2001). "Reliability Analysis of Steel Connection Components Based on FEM." *Engineering Failure Analysis*, Vol. 8, No. 1, pp 29-48.

Newland, D. E. and Cebon, D. (2002). "Could the World Trade Center have been Modified to Prevent Its Collapse?" *Journal of Engineering Mechanics*, Vol. 128, No. 7, pp 795-800.

Nickitopoulou, A., Protopsalti, K. and Stiros, S. (2006). "Monitoring Dynamic and Quasi-static Deformations of Large Flexible Engineering Structures with GPS: Accuracy, Limitations and Promises." *Engineering Structures*, Vol. 28, No. 10, pp 1471-1482.

Nordenson, G. (2003.). "Tall Buildings." *Museum of Modern Art*, New York.

Ogaja, C., Rizos, C. and Han, S. W. (2000). "Is GPS Good Enough for Monitoring the Dynamics of High-rise Buildings?" *2nd Trans Tasman Surveyors Congress*, Queenstown, New Zealand, 20-26 August, pp 150-164.

Ogaja, C., Rizos, C., Wang, J. and Brownjohn, J., (2001). "A Dynamic GPS System for On-line Structural Monitoring." *Int. Symp. on Kinematic Systems in Geodesy, Geomatics & Navigation (KIS2001)*, Banff, Canada, 5-8 June, pp 290-297.

Olsen, P. C. (2001). "Rigid-plastic Finite Element Analysis of Steel Plates, Structural Girders and Connections." *Computer Methods in Applied Mechanics and Engineering*, Vol. 191, No. 8-10, pp 761-781.

Onur, T., Ventura, C. E. and Finn, W. D. L. (2004). "Effect of Earthquake Probability Level on Loss Estimation." *13th World Conference on Earthquake Engineering*, Vancouver, B. C., Canada.

Pantelides, C. P., Truman, K. Z. Behr, R. A. and Belarbi, A. (1996). "Development of a Loading History for Seismic Testing of Architectural Glass in a Shop-front Wall System." *Engineering Structures*, Vol. 18, No. 12, pp 917-935.

Phan, L. T. and A. W. Taylor (1996). "State of the Art Report on Seismic Design Requirements for Nonstructural Building Components." *US National Institute of Standards and Technology Interim Report*.

Pink, B. (2008). "2008 Year Book Australia." *Australian Bureau of Statistics*, Canberra, Australia.

Porcaro, R., Hanssen, A. G. Langseth, M. and Aalberg, A. (2006). "The Behaviour of a Self-piercing Riveted Connection under Quasi-static Loading Conditions." *International Journal of Solids and Structures*, Vol. 43, No. 17, pp 5110-5131.

Raftoyiannis, I. G. (2005). "The Effect of Semi-rigid Joints and an Elastic Bracing System on the Buckling Load of Simple Rectangular Steel Frames." *Journal of Constructional Steel Research*, Vol. 61, No. 9, pp 1205-1225.

Ravi Kumar, G., Satish Kumar, S.R. and Kalyanaraman, V. (2007). "Behaviour of Frames with Non-Buckling Bracings under Earthquake Loading." *Journal of Constructional Steel Research*, Vol. 63, No. 2, pp 254-262.

Rodrigues, F. C., Saldanha, A. C. and Pfeil, M. S. (1998). "Non-linear Analysis of Steel Plane Frames with Semi-rigid Connections." *Journal of Constructional Steel Research*, Second World Conference on Steel in Construction, Vol. 46, No. 1-3, pp 94-97.

Sabelli, R., Mahin, S. and Chang, C. (2003). "Seismic Demands on Steel Braced Frame Buildings with Buckling-Restrained Braces." *Engineering Structures*, Vol. 25, No. 5, pp 655-666.

Sabnis, G. M. (1983). "Structural Modeling and Experimental Techniques." *Prentice-Hall*, Englewood Cliffs, N.J..

Sahoo, N., Mahapatra, D. R. Jagadeesh, G., Gopalakrishnan, S. and Reddy, K.P.J. (2007). "Design and Analysis of a Flat Accelerometer-based Force Balance System for Shock Tunnel Testing." *Measurement*, Vol. 40, No. 1, pp 93-106.

Sarma, K. C. and Adeli, H. (2005). "Comparative Study of Optimum Designs of Steel High Rise Building Structures Using Allowable Stress Design and Load and Resistance Factor Design Codes." *Practice Periodical on Structural Design and Construction*, Vol 10, No. 1, pp 12-17.

Schneider, S. P. and Alostaz, Y. M. (1998). "Experimental Behavior of Connections to Concrete-filled Steel Tubes." *Journal of Constructional Steel Research*, Vol. 45, No. 3, pp 321-352.

Seco, A., Tirapu, F. Ramirez, F., Garcia, B. and Cabrejas, J. (2007). "Assessing Building Displacement with GPS." *Building and Environment*, Vol. 42, No. 1, pp 393-399.

Sev, A. (2001). "Integrating Architecture and Structural Form in Tall Steel Building Design." *CTBUH Review*, Vol. 1, No. 2.

Shakourzadeh, H., Guo, Y. Q. and Batoz, J. L. (1999). "Modeling of Connections in the Analyses of Thin-walled Space Frames." *Computers & Structures*, Vol. 71, No. 4, pp 423-433.

Shanmugam, N. E., Yu, C. H. Liew, J. Y. R. and Teo, T. H. (1998). "Modelling of Steel-Concrete Composite Joints." *Journal of Constructional Steel Research, Second World Conference on Steel in Construction*, Vol. 46, No. 1-3, pp 247-249.

Shih, V. (2008). "The Republic of China Year Book 2008." *Government Information Office*, Republic of China.

Simoès da Silva, L. and Girao Coelho, A. M. (2001). "An Analytical Evaluation of the Response of Steel Joints under Bending and Axial Force." *Computers & Structures*, Vol. 79, No. 8, pp 873-881.

Stafford Smith, B. and Coull, A. (1991). "Tall Building Structures : Analysis and Design." *Wiley*, New York.

Standards Australia (1992). "AS/NZS 4063: 1992 Timber Stress Graded-In-grade Strength and Stiffness Evaluation." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2001). "AS 3600: 2001 Concrete Structures." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2002). "AS/NZS 1170.0: 2002 Structure Design Actions-General Principles." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2002). "AS/NZS 1170.2: 2002 Structure Design Actions-Wind Actions." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2003). "AS 3850: 2003 Tilt-up Concrete Construction." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2004). "AS/NZS 2269: 2004 Plywood-Structural." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2006). "AS/NZS 2098: 2006 Methods of Test for Veneer and Plywood-General Introduction and List of Methods." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2007). "AS 1170.4: 2007 Structure Design Actions-Earthquake Actions in Australia." *Standards Australia/ Standards New Zealand*, Australia.

Standards Australia (2008). "AS/NZS 1554: 2008 Structural Steel Welding Set." *Standards Australia/ Standards New Zealand*, Australia.

Strelitz, Z. (2005). "Tall Buildings: a Strategic Design Guide." *Riba*, London.

Su, R. K. L., Chandler, A. M., Sheikh, M. N. and Lam, N. T. K. (2005). "Influence of Non-structural Components on Lateral Stiffness of Tall Buildings." *The Structural Design of Tall and Special Buildings*, Vol. 14, No. 2, pp 143-164.

Sugiyama T., Uemura, M., Fukutama, H., Nakano, K. and Matsuzaki, Y. (2000). "Experimental Study on the Performance of the RC Frame Infilled Cast-in-place Non-structural RC Walls Retrofitted by Using Carbon Fibre Sheets." *Proceedings of the 12th WCEE*, Auckland, New Zealand, Paper No 2153.

Tamura, Y., Kawai, H., Uematsu, Y., Marukawa, H., Fujii, K. and Taniike, Y. (1996). "Wind Load and Wind-induced Response Estimations in the Recommendations for Loads on Buildings, AIJ 1993." *Engineering Structures*, Vol. 18, No. 6, pp 399-411.

Taranath, B. S. (1998). "Steel, Concrete, and Composite Design of Tall Buildings." *McGraw-Hill*, New York.

Turvey, G. J. and Brooks, R. J. (2002). "Semi-rigid--Simply Supported Shear Deformable Pultruded GRP Beams Subjected to End Moment Loading: Comparison of Measured and Predicted Deflections." *Composite Structures*, Vol. 57, No. 1-4, pp 263-277.

Vigh, L. G. and Dunai, L. (2004). "Finite Element Modelling and Analysis of Bolted Joints of 3D Tubular Structures." *Computers & Structures, Computational Structures Technology*, Vol. 82, No. 23-26, pp 2173-2187.

Villaverde, R. (2000). "Implementation Study of Aseismic Roof Isolation System in 13-Story Building." *JSEE: Spring 2000*, Vol. 2, No. 2, pp17-27.

Wald, F., Simoes da Silva, L., Moore, D. B., Lennon, T., Chladna, M., Santiago, A., Benes, M. and Borges, L. (2006). "Experimental Behaviour of a Steel Structure under Natural Fire." *Fire Safety Journal*, Vol. 41, No. 7, pp 509-522.

Wan-Shin, P. and Hyun-Do, Y. (2006). "Panel Shear Strength of Steel Coupling Beam-wall Connections in a Hybrid Wall System." *Journal of Constructional Steel Research*, Vol. 62, No. 10, pp 1026-1038.

Wilkinson, S. M. and Hiley, R. A. (2006). "A Non-linear Response History Model for the Seismic Analysis of High-rise Framed Buildings." *Computers & Structures*, Vol. 84, No. 5-6, pp 318-329.

Wu, J. C., Lu, W. C. and Hsu, W. C. (2006). "Implementation of a Feasible Control Design Process Incorporating Robustness Criteria for Wind-Excited High-Rise Buildings." *Journal of Structural Engineering*, Vol. 132, No. 1, pp 89-101.

Xie, Q. (2005). "State of the Art of Buckling-restrained Braces in Asia." *Journal of Constructional Steel Research*, Vol. 61, No. 6, pp 727-748.

Xiong, X., Zou, Q. Lu, D. and Wang, W. (1998). "Balance-approach for Load-displacement Measurement of Microstructures." *Mechatronics*, Vol. 8, No. 5, pp 549-559.

Yu, W. K., Chung, K. F. and Wong, M. F. (2005). "Analysis of Bolted Moment Connections in Cold-formed Steel Beam-column Sub-frames." *Journal of Constructional Steel Research*, Vol. 61, No. 9, pp 1332-1352.

Zaharia, R. and Dubina, D. (2006). "Stiffness of Joints in Bolted Connected Cold-formed Steel Trusses." *Journal of Constructional Steel Research*, Vol. 62, No.3, pp240-249.

"Historical World Maps." <<http://world-map.nl>>, (20 Jan, 2009).

"Introduction of Australia." <<http://www.australia.gov.au/>> (21 Jan, 2009).

"Introduction of Dalian, China." *Dalian, China*, <<http://www.dl.gov.cn/english>> (20 Jan, 2009).

"Introduction of Tianjin, China." *Tianjin Municipal People's Government* <<http://www.tj.gov.cn/english>> (20 Jan, 2009).

MapXL, Inc. "China Political Map." <<http://www.chinamapxl.com/political-map.html>> (20 Jan, 2009).

Reference

“Taipei 101, The World Tallest Tower.” <<http://taipei-101.info>> (20 Jan, 2009).

“Weather Conditions of Australia.” *Australian Government Bureau of Meteorology*, <<http://www.bom.gov.au/lam/climate/levelthree/ausclim/ausclim.htm>> (22 Jan, 2009).

APPENDICES

Appendix I: Published Papers

Appendix II: Design of Bolts, Pin Connections and Welds from AS 4100 (Section 9)

APPENDIX I

PUBLISHED PAPERS

- Li, B., Hutchinson, G. L. and Duffield, C. F. (2009). “The Influence of Non-structural Components on Tall Building Stiffness.” *The Structural Design of Tall and Special Buildings*, DOI: 10.1002/tal.565, In Press.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2009). “The Influence of Non-structural Components on the Serviceability Performance of High-rise Buildings.” *Australian Journal of Structural Engineering*, Vol.10, No. 1, pp. 53-62.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “Quantification of the Contribution of Non-Structural Components to the Performance of High-rise Buildings.” *Proceedings of EASEC-11*, Taipei, Taiwan.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “A Parametric Study of the Lateral Performance of a High-rise Structure.” *Proceedings of ASEC 2008*, Melbourne, Australia.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2008). “Simplified Finite Element Modelling of Multi-storey Buildings: The Use of Equivalent Cubes.” *Electronic Journal of Structural Engineering*, Vol.8, pp. 40-45.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2007). “Field Investigation Report - Analysis of Connection Properties.” *Department of Civil & Environmental Engineering Research Report*, RR/Struc/11/2006, The University of Melbourne.
- Li, B., Duffield, C. F. and Hutchinson, G. L. (2007). “The Influence of Structural and Non-structural Components on the Lateral Performance of High-rise Buildings.” *Proceedings of IESC-4*, Melbourne, Australia.
- Li, B., Kealy, A., Duffield, C. F. and Hutchinson, G. L. (2007). “Evaluating the Performance of Low-Cost Inertial Sensors for use in Integrated Positioning Systems.” *IGNSS2007*, Sydney, Australia.
- Hutchinson, G. L., Collier, P., Duffield, C. F., Gad, F. E., Li, B. and Mendis, P. A. (2006). “Integration of GPS Measurement and Secondary Structural Elements into

the Serviceability Design of Structures.” *Proceedings of EASEC-10*, Bangkok, Thailand.

THE INFLUENCE OF NON-STRUCTURAL COMPONENTS ON TALL BUILDING STIFFNESS

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SUMMARY

The lateral load resisting system of a multi-storey building is considered to be an assembly of structural components, such as the structural frame, shear walls, concrete cores, etc. However, in reality, some so-called 'non-structural components (NSCs)' also play important roles in adding stiffness to the building. To evaluate the contributions from those NSCs and to quantify some of their contributions to the stiffness of the structure under service level loads, this paper reports on the analysis of a lateral load resisting system with different components so that the stiffness contribution from each individual component may be evaluated. Results from finite element analyses are verified by other theoretical calculations. Discussions and conclusions on the performance of both single components and the building system are also provided. Copyright © 2009 John Wiley & Sons, Ltd.

1. INTRODUCTION

The lateral load resisting system of a multi-storey building is normally considered to be an assembly of some of the structural components, such as the structural frame, shear walls, concrete cores, etc. The role played by the so-called 'non-structural components (NSCs)' is not incorporated in either Australian or international standards, and is therefore not considered in the current design process. However, more and more evidence has indicated that the structural role of NSCs, in resisting lateral loads, can be very significant, and the interaction between the NSCs and the structural skeleton may lead to distress, loss of serviceability and occasional failure of the NSCs (e.g., Arnold, 1991; Melchers, 1989; Hall, 1995; Phan and Taylor, 1996; Naeim, 1999 and McDonnell, 2001). The actual performance of real buildings differs significantly from that of idealized structural models (Naeim, 1999; Sugiyama *et al.*, 2000; Hutchinson *et al.*, 2006). Gad *et al.* (1998, 1999a, 1999b, 2000) have clearly shown that NSCs in low-rise buildings can increase the building's lateral stiffness and strength by more than 100%. This accounts for the difference between the theoretical estimates and real performance. In multi-storey buildings, most designers of partitions and facades opt for the theoretical approach of complete detachment of these components (i.e., they assume that cladding and partitions do not contribute to the lateral stiffness of the structure). In practice, this would rarely be the case even when gaps are specified. The practicalities of building construction result in the inevitable transfer of forces from NSCs to the skeletal structure and vice versa (Arnold, 1991; Freeman, 1977). This has often resulted in serviceability damage to the NSCs, even after moderate wind or earthquake events. In order to better understand the role played by the NSCs in influencing structural performance, it is necessary to analyse and evaluate the contribution of each component to the overall lateral performance of multi-storey buildings. The purpose of this paper is to systematically quantify the stiffness

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contributions from different components of a multi-storey building system, especially contributions from NSCs, so that the significance of those NSCs can be identified.

2. DESCRIPTION OF ANALYSIS

This paper introduces finite element analyses of the lateral stiffness of different assemblies of building components under service loads. Theoretical calculations are also provided in order to establish the reliability of the finite element models.

2.1 Assumptions and limitations

To simplify and generalize the analyses, some assumptions have to be made:

- (1) Structural system: the structure is assumed to be symmetrical.
- (2) Materials: any plastic/non-elastic behaviour of materials is excluded. The contribution of any steel reinforcement in the concrete material is not included.
- (3) Loading conditions: static lateral point loads with the magnitude within the serviceability limit, were applied to the beam-column joints along one face of the structure.
- (4) No dynamic characteristics are included in this study.

2.2 Structural and non-structural components

The multi-storey system involved in this analysis includes the following components: a structural frame, coupled shear walls, infill walls, facade panels, doors and windows. Five different combinations of those elements were analysed in order that the stiffness contributions of each component can be clearly identified. These combinations are:

- (1) Bare frame (Figure 1(a))
- (2) Frame with infill walls (Figure 1(b))
- (3) Coupled shear wall (shear walls with openings) (Figure 1(c))
- (4) Coupled shear wall with doors and windows
- (5) Shear-wall frame with facade panels (Figure 1(d))

This study is based on a case-study building that has a $30\text{ m} \times 30\text{ m}$ floor plan with main column supports at every 10 m (Figure 1(a)). The total height of the building is 90 m, with a storey height of 3 m over 30 storeys. To simplify the analysis, a uniform cross-section steel beam 360UB56.7 was assigned to the beams and a steel 310UC118 column to the columns. The thickness of the coupled shear wall is 0.4 m (Figure 1(c)). The 0.1 m precast concrete panel was introduced in the analysis as infill walls (shown in Figure 1(b)). The gap between the frame and the infill wall is very small and no special bonds or connections were made between the infill walls and the structural frame, only contact elements were defined in the finite element model to assure the contact between the frame and infill walls. The facade system is composed of aluminium facade frames and glass panels (Figure 1(d)). The cross-sectional area of an aluminium facade frame is $0.1\text{ m} \times 0.03\text{ m}$. The thickness of the glass panels is 0.02 m. The windows are considered to have 0.01 m thick glass panels with $0.1\text{ m} \times 0.05\text{ m}$ aluminium frames. Details of the geometric and material properties of these structural and non-structural components are listed in Table 1.

When doing an analysis for assemblies of components, different parts of the structural frame were chosen in order to identify the contributions from various NSCs. For example, in the analysis of the infill wall frame performance, a single-span multi-level and multi-bay 3-D frame (Figure 1(b)) was

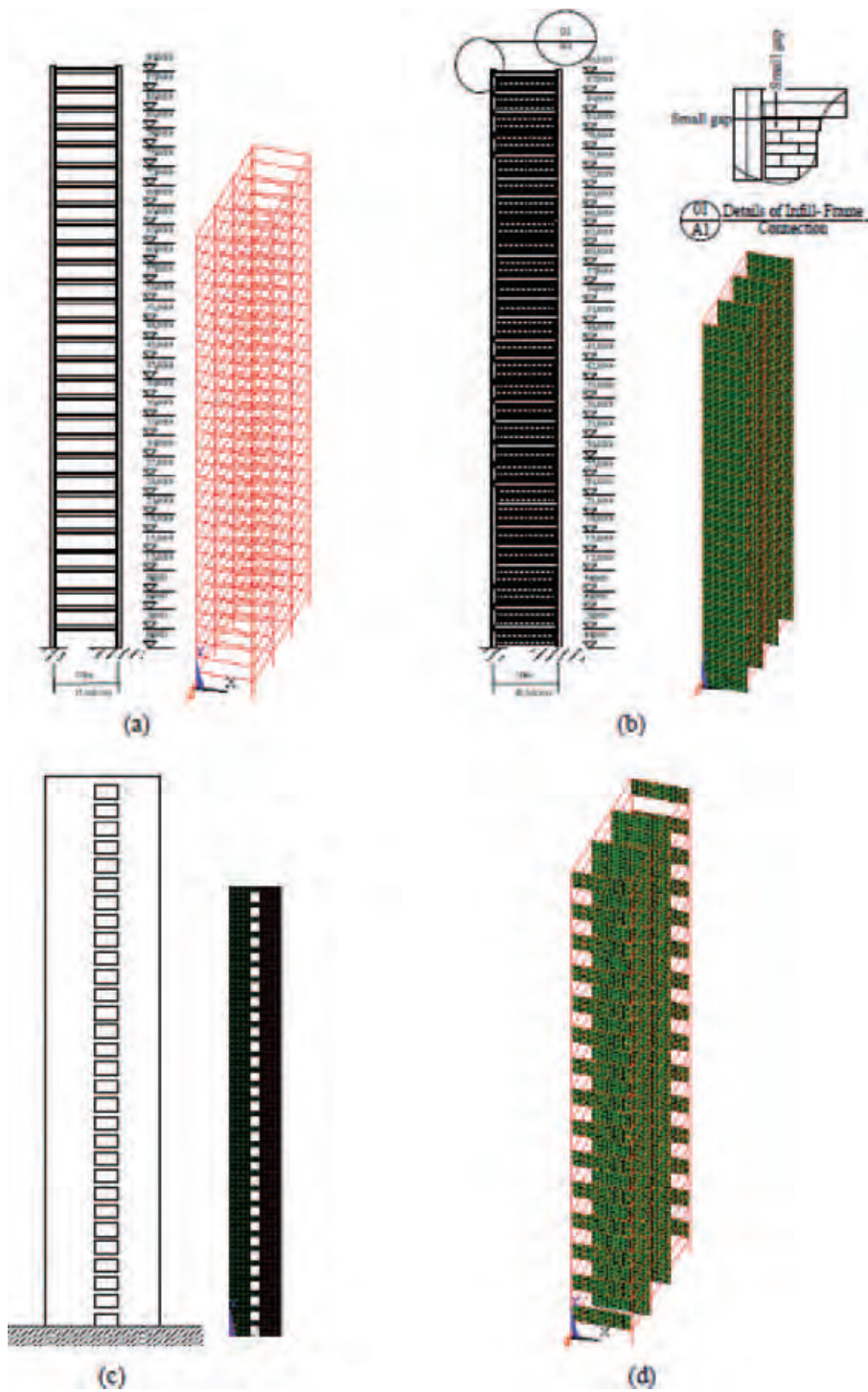








Figure 1. Configurations of the structure: (a) bare frame; (b) frame with infill walls; (c) coupled shear wall; (d) shear wall frame with façade

Table 1. Element and material details

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column	360UB56.7	Steel	Linear Elastic $E = 2.0 \times 10^{11}$ Pa $\mu = 0.29$	BEAM4
Beam	310UC118		$density = 7850$ kg/m ³	BEAM4
Shear Wall	 0.4m	Concrete	Linear Elastic $E = 2.5 \times 10^{10}$ Pa $\mu = 0.15$	SHELL63
Infill Wall	 0.1m		$density = 2400$ kg/m ³	PLAN42
Façade Panel	 0.02m	Glass	Viscoelastic $G_0 = 2.74 \times 10^{10}$ Pa $G_b = 6.05 \times 10^{10}$ Pa $1/\beta = 0.53$	SHELL63
Window Panel	 0.01m			
Façade Frame	 0.1m 0.03m	Aluminium	Linear Orthotropic $E_x = 3.07 \times 10^{11}$ Pa $E_y = 3.58 \times 10^{11}$ Pa $E_z = 3.58 \times 10^{11}$ Pa $\mu_{xy} = \mu_{yz} = \mu_{xz} = 0.2$ $G_{xy} = G_{yz} = G_{xy} = 1.269 \times 10^{11}$ Pa	BEAM4
Window Frame	 0.1m 0.05m			
Connection		--	k	CONTAC52

chosen to identify the stiffness contributions from the infill wall (Figure 1(b)). When evaluating the contributions from doors and windows, a coupled shear wall was analysed separately. Details of the configurations for every stage are discussed in the following sections.

2.3 Finite element analysis software

ANSYS10-0 from ANSYS Inc. PA. USA was used in this study to conduct the finite element analyses. The advantages of this software are that details of the structure can be well defined and any non-linearity of structural and material characteristics/behaviour can also be represented and calculated. However, considerable computational time and memory capacity is required to solve problems with the complexity of a tall building structure.

2.4 Modelling details

2.4.1 Bare frame analysis

Since any effect of the foundation is beyond the scope of this study, the total six degrees of freedom of the structure base were fully constrained in order to match with the widely adopted ‘cantilever’ theory in tall building analysis. Three-node 3-D beam elements (BEAM4) were used to represent both columns and beams. The connections between beams and columns were considered as rigid. Details of the ANSYS elements are listed in Table 1.

2.4.2 Frame with infill walls

Concrete block infill walls were involved in the analysis. To model the connection between infill walls and the frame, a very small gap was defined as shown in Figure 1(b). However, in real practice, the infill wall is normally built as a built-in wall. A four-node 3-D shell element (SHELL63) was used to represent the infill walls, and the point-to-point contact element (CONTAC52) was created to simulate the connectivity across gaps. Details of elements are listed in Table 1.

2.4.3 Shear wall frame with facade panels

In the finite element analyses, the same shell element (SHELL63) was adopted for the glass panels of the façade system. Three-D beam elements (BEAM4) were used to model the aluminium façade frame. The façade system was considered as a built-in floor-to-ceiling façade as shown in Figure 1(d). Table 1 presents the details of the elements and materials allocated in the analysis. Connections between the façade frame and the structural components are not of the interest of this study for that detailed discussions on the influence of different types of connections have been provided in another paper of the authors (Li *et al.*, 2007).

2.4.4 Coupled shear wall analysis

Concrete coupled shear walls with dimensions of 10.0 m × 90.0 m × 0.4 m was modelled by shell elements (SHELL63 (Figure 1(c) and Table 1)). Universal openings (2.0 m × 2 m × 0.4 m) were located at the middle of each span at each level of the wall (Figure 1(c)).

2.4.5 Coupled shear wall with windows

Windows were included in the second step analysis of the coupled shear wall in order that the stiffness contribution of the windows could be evaluated. The glass panels were represented by shell elements (SHELL63) and the aluminium window frames were modelled by beam elements (BEAM4) (Table 1).

3. RESULTS AND DISCUSSIONS

3.1 Bare frame analysis

The deflection at the top of a multi-storey frame under service loads can be considered as the accumulation of the storey drifts up the structure, i.e. the sum of the individual storey drifts caused by column and girder flexure. This is represented by Equations 1 to 6 below developed by Stafford Smith and Coull (1991). The nomenclature of the equations is provided at the beginning of the paper.

$$\delta_g = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L}\right)_i} \quad (1)$$

$$\delta_c = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h}\right)_i} \quad (2)$$

$$\delta_{if} = h_i A_0^i \quad (3)$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \quad (4)$$

$$\delta_i = \delta_g + \delta_c + \delta_{if} = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L}\right)_i} + \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h}\right)_i} + \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \times h_i \quad (5)$$

$$\Delta = \sum \delta_i \quad (6)$$

Figure 2 compares the results of the frame from both the theoretical calculations and the finite element analysis. Lateral loads with the sum equivalent to 48 kN were uniformly distributed to the beam column connections along one face of the structure at every level. The theoretical result and the finite element result show high consistency. When the lateral load reaches about 48 kN, the displacement predicted by the theory is 51.4 mm compared with 54.1 mm from the finite element analysis result. The difference between these two sets of results is 5%, which is satisfactory for the structure under consideration. Similarly, identical results from finite element analyses and theoretical analyses are also achieved in the storey drift analysis. Furthermore, the force–displacement relationships plotted by both the theoretical and the finite element analyses show high linearity as would be expected, given the assumptions and the linear elastic material properties.

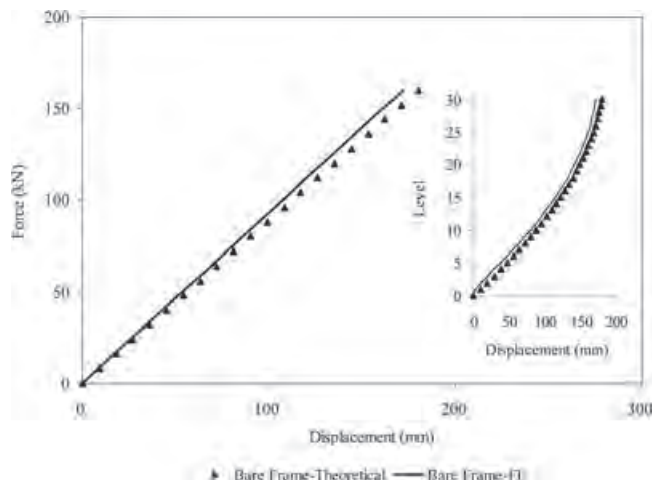


Figure 2. Force–displacement relationship and the total storey drift of bare frame analysis. FE, Finite Element

3.2 *Frame with infill walls*

According to Stafford Smith and Coull (1991), a frame with infill walls should be analysed as an equivalent bracing system. When under the service load, the top deflection of the braced frame will be the sum of the drift in each storey. The drift in storey i is a combination of deflection caused by the shear deflection of the braced bents at storey i and the flexural column and beam drifts at that storey (Equations 7 to 11).

$$\delta_s = \frac{Q_i}{2E} \left(\frac{d^3}{L_i^2 A_d} \right) \tag{7}$$

$$\delta_{if} = h_i \theta_{if} = h_i A_0^i \tag{8}$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_0 - h_{i-1/2}}{2} \tag{9}$$

$$\delta_i = \delta_s + \delta_{if} \tag{10}$$

$$\Delta = \sum \delta_i \tag{11}$$

As shown in Figure 3, a discrepancy of the force–displacement relationships exists between the theoretical analysis and the finite element analysis. This discrepancy indicates that the finite element model models the extra stiffness caused by the contact between the infill walls and the structural frame only if the contact has happened, whilst the theory used in solving the infill wall frame structure considers infill walls as braces and analyses the infill wall frame as a braced frame from the very beginning of the analysis.

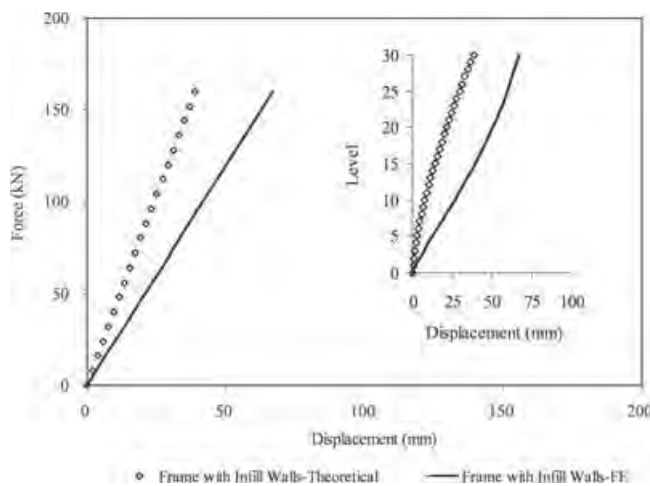


Figure 3. Force–displacement relationship and the total storey drift of frame with infill wall analysis. FE, Finite Element

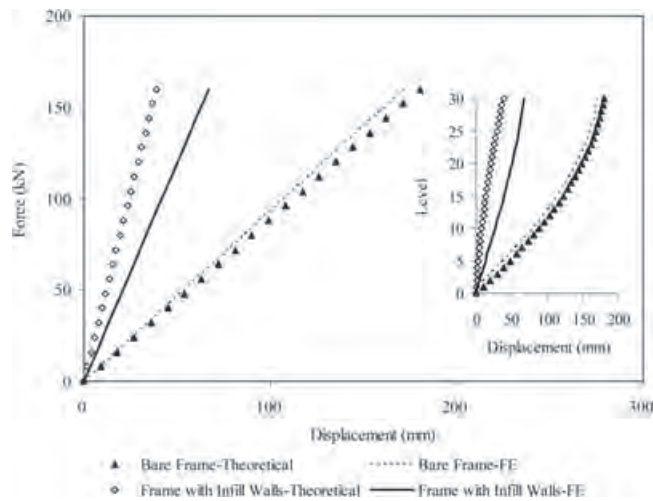


Figure 4. Comparison of the force–displacement relationship and the total storey drift of frame with and without infill walls. FE, Finite Element

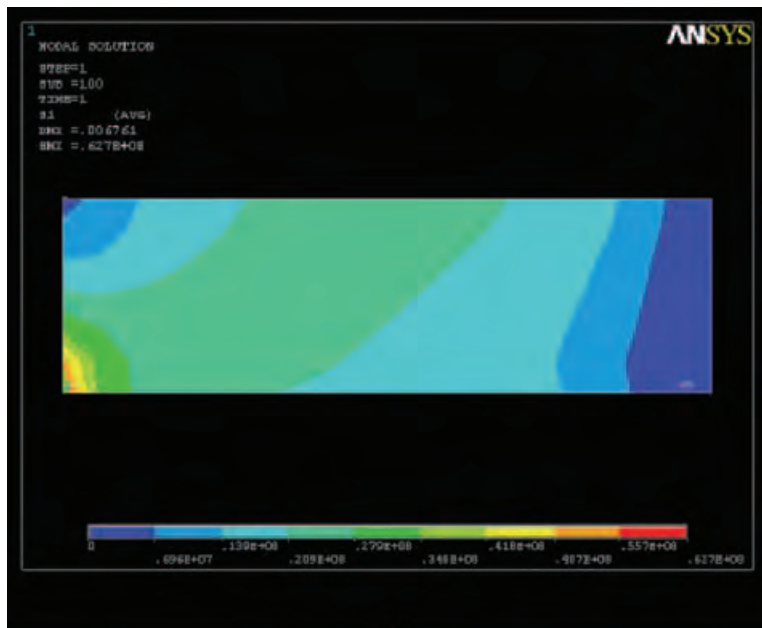
3.2.1 The stiffening effect of infill walls

By comparing the force–displacement relationships and the storey drifts of the bare frame and frame with infill walls as shown in Figure 4, it is clear that when under the same loading conditions, more than 60% increase of the structural stiffness can be obtained by including infill walls into the analysis. This leads to a consequent decrease of the top deflection and the storey drifts of the structure. This indicates that significant increase of the structural stiffness can be realized by adding infill walls to the structural frame.

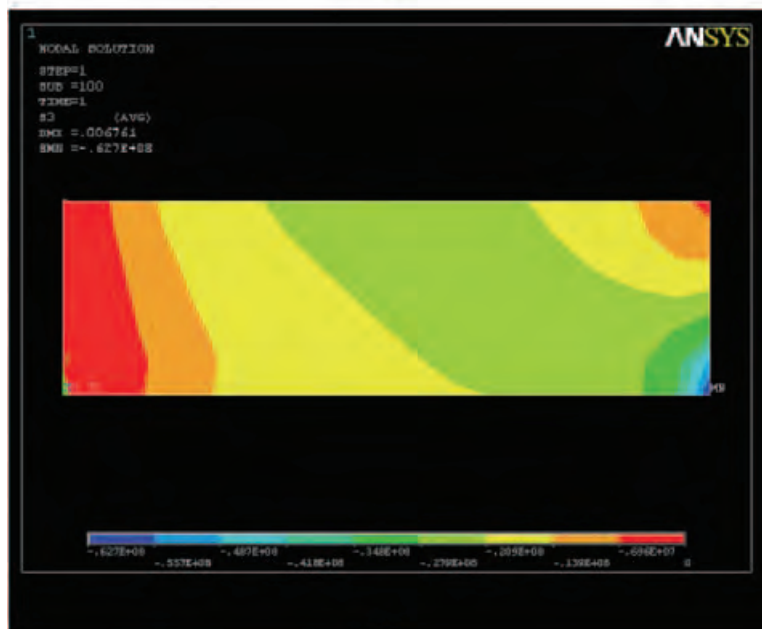
3.2.2 The stress distribution within the infill walls

Even though considerable increase of the structural stiffness can be achieved by including infill walls into the structural analysis, the load redistribution within components caused by the integration of NSCs also introduces dramatic changes to the stress distribution in the infill walls. In this analysis, the extreme scenario in which infill walls have direct contact with structural frame at all times when subject to the maximum allowable service level movement was investigated. Figure 5 plots the stress distribution within infill walls when the wall is subject to a 6-mm top deflection (the maximum allowable storey drift of this structure under the serviceability limit according to Australian Standards (AS/NZS 1170 series, 2002)). The maximum tensile and compressive stresses shown in the graph are both around 62.7 MPa which are beyond the tensile and compressive capacity ranges of commonly used construction concrete for precast panels (2 ~ 5 MPa for tensile strength and 20 ~ 60 MPa for compressive strength).

However, based on the field reconnaissance within the Asian-Pacific region, the extreme scenario depicted above seldom happens as some measures have been adopted in the practice. In Australia, gaps between infill walls and the frame are specified in the structural design and are filled using elastic materials during construction. This, to some extent, reduces the chance of direct contact between infill walls and the structural frame, thus providing a margin for the actual movement of the infill wall. Moreover, in some other countries, China for example, masonry infill walls are built in the frame with the top-layer bricks oriented along an in-plane 45° diagonal line (Figure 6). By doing this, the stress transferred from the frame to the infill walls can be effectively dispelled.



(a)



(b)

Figure 5. Stress distribution within the infill wall when subject to the maximum allowable storey drift according to Australian standards (AS/NZS 1170 series 2002): (a) tensile stress; (b) compressive stress (unit: Pa)

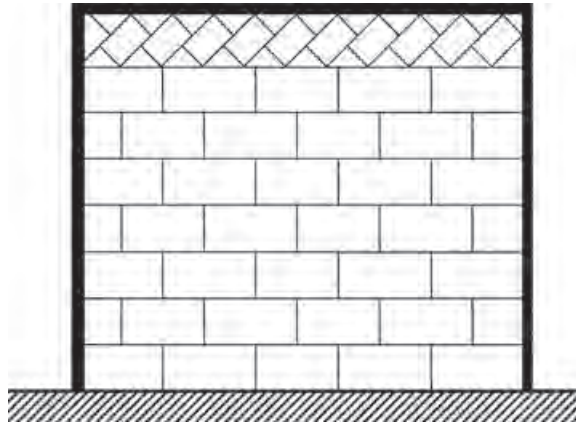


Figure 6. Simple demonstration of the practice used for dissipating the load transferred from the frame to infill walls in China

Moreover, the out-of-plane behaviour of the infill panels, such as the buckling issue, has not been covered in this study. It is understandable that with a high slenderness ratio (in this study, 30:1), the infill walls will tend to buckle under the combination of gravity loads and the out-of-plane loads. Under these circumstances, the contribution of infill walls to the structural stiffness will be diminished and the algorithm of the analysis needs to be revised. However, this study only focuses on the serviceability of the structure and the in-plane behaviour of infill walls rather than its out-of-plane behaviour. Taking into account the practices introduced above in different countries, the opportunity for the infill wall to buckle under this circumstance is slim. Thus, the out-of-plane behaviour of infill walls is beyond the scope of this study.

3.3 Coupled shear wall analysis

Based on the theory from Stafford Smith and Coull (1991) (Equations 12 to 22), a coupled shear wall under lateral loads acts as a pure cantilever. Figure 7 shows the results from both the theoretical and finite element analyses of the coupled shear wall. When under 120 kN lateral load, top deflections of the coupled shear wall under theoretical and finite element analyses are 13.7 mm and 14.7 mm, respectively. The difference between those two sets of results is less than 7%, which validates the finite element coupled shear wall model involved in this study.

$$y = \frac{wH^4}{EI} \left[\frac{1}{24} \left(\left(1 - \frac{z}{H} \right)^4 + 4 \times \frac{z}{H} - 1 \right) + \frac{1}{k^2} \left(\left(\frac{1}{2 \times (k\alpha H)^2} \right) \times \left(2 \times \frac{z}{H} - \left(\frac{z}{H} \right)^2 \right) - \frac{1}{24} \times \left(\left(1 - \frac{z}{H} \right)^4 + 4 \times \frac{z}{H} - 1 \right) - \frac{1}{(k\alpha H)^4 \cosh k\alpha H} \times \left(1 + k\alpha H \sinh k\alpha H - \cosh k\alpha z - k\alpha H \sinh k\alpha(H - z) \right) \right] \right] \quad (12)$$

$$y_{\max} = \frac{wH^4}{8EI} F_3(k, \alpha H) \quad (13)$$

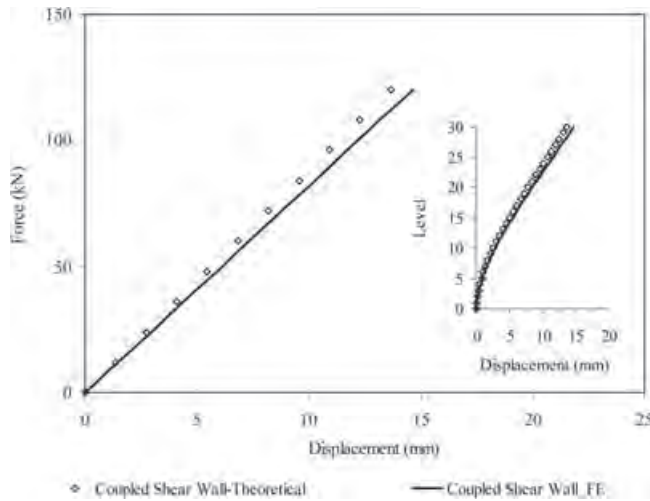


Figure 7. Force–displacement relationship and the total storey drift of coupled shear wall analysis. FE, Finite Element

$$F_3 = 1 - \frac{1}{k^2} \left(1 - \frac{4}{(k\alpha H)^2} + \frac{8}{(k\alpha H)^4 \cosh k\alpha H} (1 + k\alpha H \sinh k\alpha H - \cosh k\alpha H) \right) \quad (14)$$

$$k^2 = 1 + \frac{AI}{A_1 A_2 l^2} \quad (15)$$

$$\alpha^2 = \frac{12I_c l^2}{b^3 h I} \quad (16)$$

$$I_c = \frac{I_b}{1+r} \quad (17)$$

$$r = \frac{12EI_b \lambda}{b^2 GA} \quad (18)$$

$$\lambda = 1.2 \quad (19)$$

$$I = I_1 + I_2 \quad (20)$$

$$A = A_1 + A_2 \quad (21)$$

$$G = \frac{E}{1+\gamma} \quad (22)$$

3.4 Coupled shear wall with windows

Doors and windows are non-structural components similar to infill walls which can enhance the in-plane stiffness of the coupled shear wall. However, because of installation techniques, material prop-

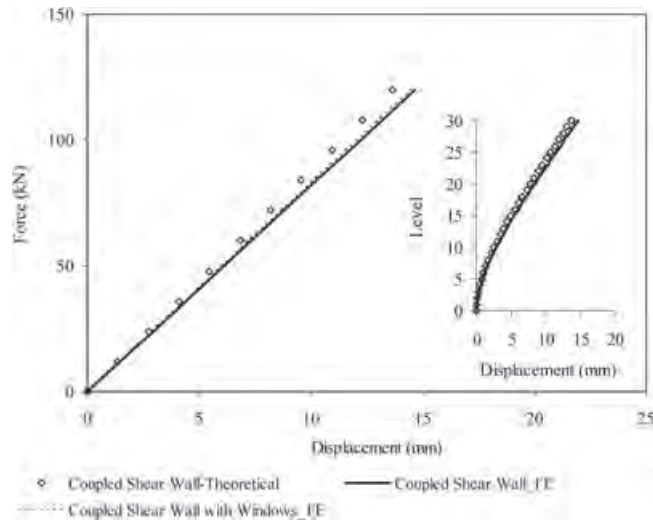


Figure 8. Comparison of the force–displacement relationship and the total storey drift of coupled shear wall with and without windows. FE, Finite Element

erties, etc., the bracing effect from doors and windows are not as quantifiable as that of infill walls. According to the reviewed literature mentioned in section 1 of this paper, limited information is available in detailing the performance of coupled shear walls with doors and windows. Consequently, only computer models were developed and analysed in this section.

3.4.1 The stiffening effect of windows

Figure 8 shows the comparison of the stiffness of a coupled shear wall with and without window panels. It is clear that even though it is not significant, windows in the coupled shear wall help reduce the storey drift of the structure by around 1.2%.

3.4.2 The stress distribution within the window panels

Though the stiffening effect of windows is not significant, it is still important to investigate the stress distribution within the window panels because the integration of windows panels into the structural analysis will lead to a load redistribution within different elements, thus possibly overstressing the NSCs which had been considered ‘non-load bearing components’.

Figure 9 plots the stress distribution within the window panel when it is subjected to the compliant drift of the allowable building storey drift according to Australian Standards (AS/NZS 1170 series, 2002). From the graph, the maximum tensile stress in the glass panel is 1.65 MPa. This is well within the range of the tensile capacity of the glass (27–62 MPa).

3.5 Shear wall frame

This analysis sets the benchmark of the analysis of shear wall frames with façade panels. The theory is adopted from Stafford Smith and Coull (1991). Based on structural theory, the deflection of the wall-frame structure can be calculated by considering that the frame and walls are working together to resist the lateral loads (Equations 23 to 25)

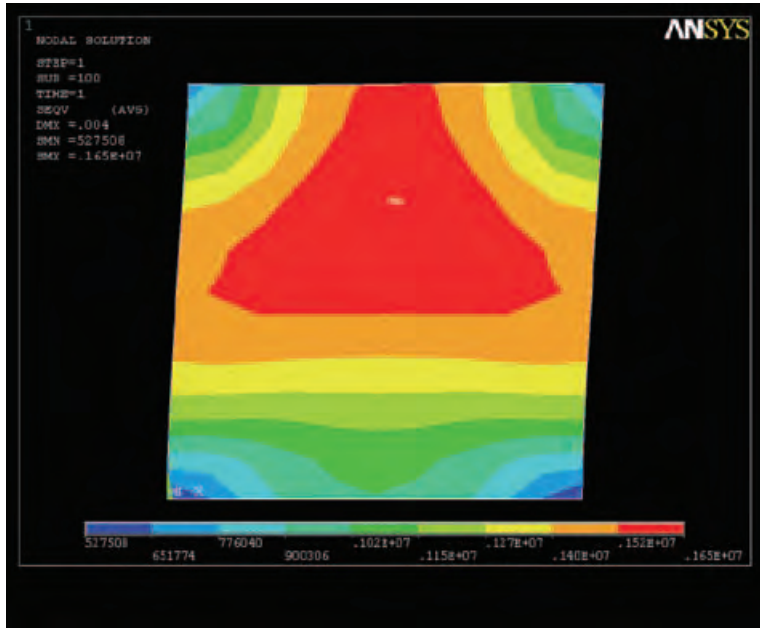


Figure 9. Stress distribution within the window panel when subject to the maximum allowable storey drift according to Australian Standards (AS/NZS 1170 series 2002) (unit: Pa)

$$y(z) = \frac{wH^4}{8EI} \left\{ \frac{8}{(\alpha H)^4} \left[\frac{\alpha H \sinh \alpha H + 1}{\cosh \alpha H} (\cosh \alpha z - 1) - \alpha H \sinh \alpha z + (\alpha H)^2 \left(\frac{z}{H} - \frac{1}{2} \left(\frac{z}{H} \right)^2 \right) \right] \right\} \quad (23)$$

$$\alpha^2 = \frac{GA}{EI} \quad (24)$$

$$GA = \frac{12E}{h_i \left(\frac{1}{G} + \frac{1}{C} \right)} = \frac{12E}{h \left(\frac{L_i}{\sum (I_g)_i} + \frac{h_i}{\sum (I_c)_i} \right)} \quad (25)$$

Figure 10 shows the force–displacement relationships and the storey drifts of the shear wall frame from both theoretical and finite element analysis. The results are close to each other, with less than 10% difference.

3.6 Shear wall frame with facade panels

In this analysis, the facade system is considered as a built-in façade, with glass panels framed by the aluminium mullions and jambs. Other types of façade system were also analysed, and the results were published elsewhere (Li *et al.*, 2009).

According to Hoenderkamp and Snijder (2003), shear wall frames with façade systems can be considered to have an outrigger system (Figure 11). Based on the theory of outrigger systems developed by Stafford Smith and Coull (1991), Equations 26 to 35 can be adopted to evaluate the performance of multi-storey shear wall frame structures with multi-storey facade systems.

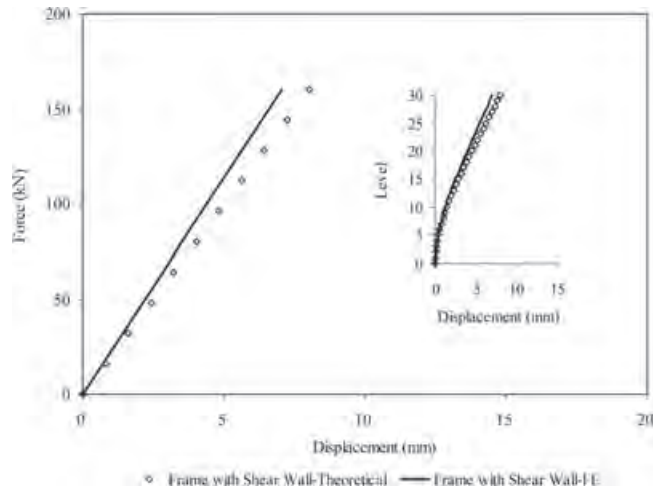


Figure 10. Force–displacement relationship and the total storey drift of frame with shear wall analysis. FE, Finite Element

$$\Delta_0 = \frac{wH^4}{8EI} - \frac{1}{2EI} \sum_{i=1}^n M_i(H^2 - x_i^2) \quad (26)$$

$$\begin{bmatrix} M_1 \\ M_2 \\ \vdots \\ M_i \\ \vdots \\ M_n \end{bmatrix} = \frac{w}{6EI} \begin{bmatrix} S_1 + S(H - X_1) & S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ S(H - X_2) & S_1 + S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_i) & S(H - X_i) & \cdots & S_1 + S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_n) & S(H - X_n) & \cdots & S(H - X_n) & \cdots & S_1 + S(H - X_n) \end{bmatrix}^{-1} \begin{bmatrix} H^3 - X_1^3 \\ H^3 - X_2^3 \\ \vdots \\ H^3 - X_i^3 \\ \vdots \\ H^3 - X_n^3 \end{bmatrix} \quad (27)$$

$$S = \frac{1}{EI} + \frac{2}{d^2(EA)_c} \quad (28)$$

$$S_1 = \frac{d}{12(EI)_0} \quad (29)$$

$$y_{\max} = \frac{wH^4}{8EI_w} - \frac{w(H^3 - x^3)(H^2 - x^2)}{12EI_w} \left\{ \frac{H}{EI_w \{S(H - x) + HS_r\}} \right\} \quad (30)$$

$$M_i = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \left(\frac{1}{EI_w} + \frac{2}{EA_c \ell^2} \right) (H - x) + \left(\frac{\ell}{12EI_f} + \frac{1}{h_i GA} \right) \right\}^{-1} \quad (31)$$

$$\omega = \frac{S_r}{S} = \left\{ \frac{\ell}{EI_r} + \frac{1}{h_i GA_r} \right\} \left\{ \frac{H}{EI_w} + \frac{H}{EA_c^2} \right\}^{-1} \quad (32)$$

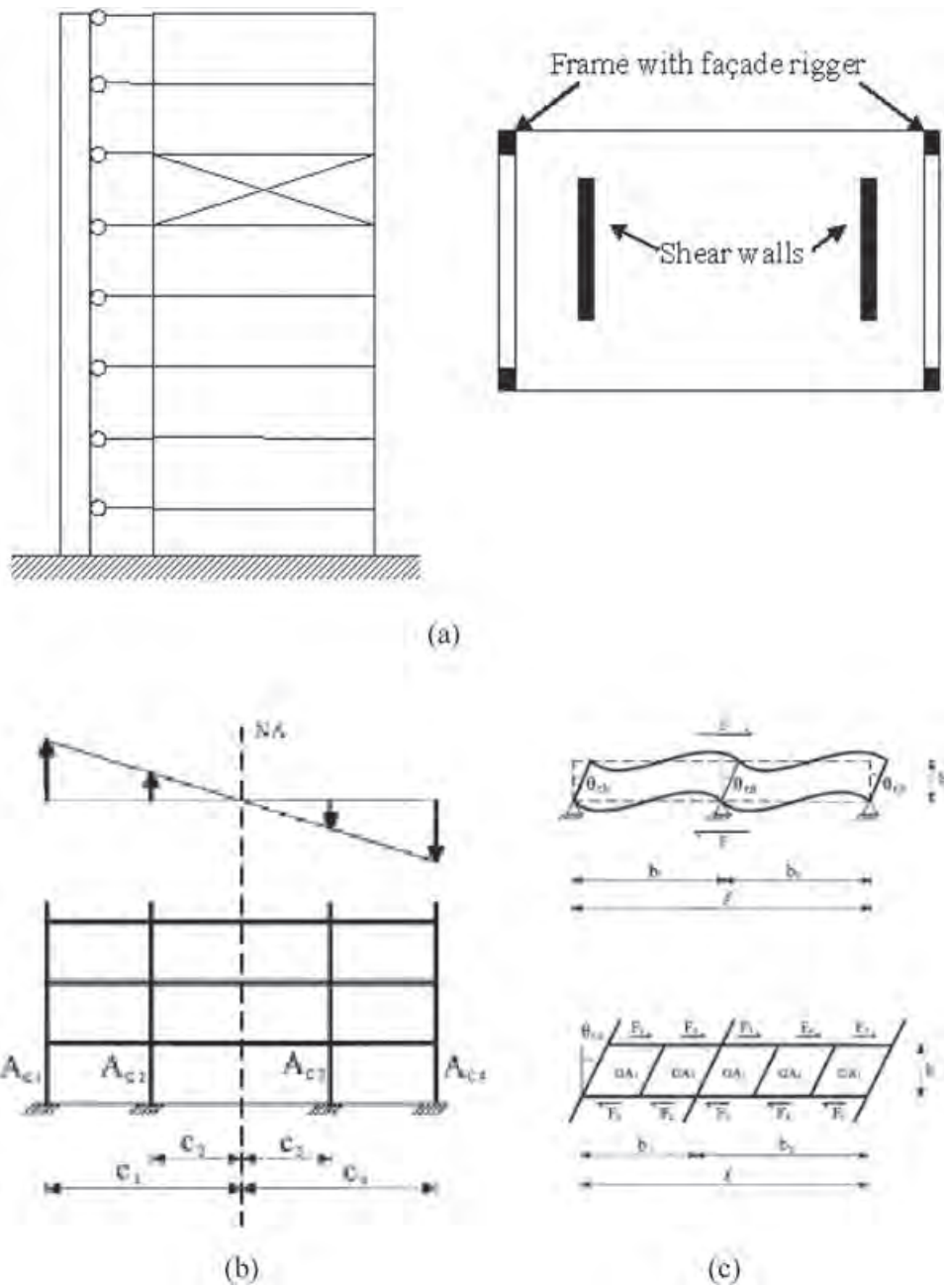


Figure 11. Shear wall frame with façade (Hoenderkamp and Snijder, 2003): (a) structural model and floor plan; (b) axial column deformation; (c) bending and shear deformation

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \frac{H}{S(H-x) + HS_r} \right\} \quad (33)$$

$$S = \frac{H}{EI_w} + \frac{H}{EAc^2} \quad (34)$$

$$S_r = \frac{\ell}{12EI_r} + \frac{1}{h_tGA_r} \quad (35)$$

Figure 12 indicates the final results from both theoretical analyses and finite element analyses of the shear wall frame with the facade system. It may be observed that the difference between the theoretical and finite element analysis results is up to 22% at a load of 160 kN. Moreover, the structure under theoretical analysis seems less stiff than it was under finite element analysis. This can be explained by analyzing the theory presented by Stafford Smith and Coull (1991), which considered the shear wall frame with the facade as an outrigger system. In this case, shear stiffness from the facade and the capacity of resisting flexural deflection from the aluminium facade frame have not been taken into account.

Hoenderkamp and Snijder (2003) did improve the theory for analysing a structure with a facade system. However, in reality, the complexity of different facade systems from connections to facade assemblies is a limitation that makes it extremely difficult to simply utilize one theory, especially for multi-storey buildings with multi-storey facade systems.

3.6.1 Stiffness contributions of the façade

Figure 13 compares the stiffness of shear wall frames with and without façades. According to the finite element results, it is clear that with the inclusion of the facade system into the structural analysis, a 12% increase of structural stiffness can be achieved. If compared with another type of facade

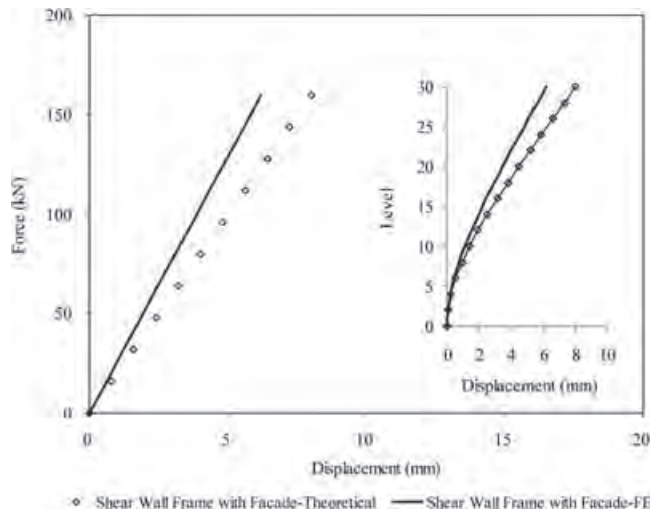


Figure 12. Force–displacement relationship and the total storey drift of shear wall frame with facade analysis. FE, Finite Element

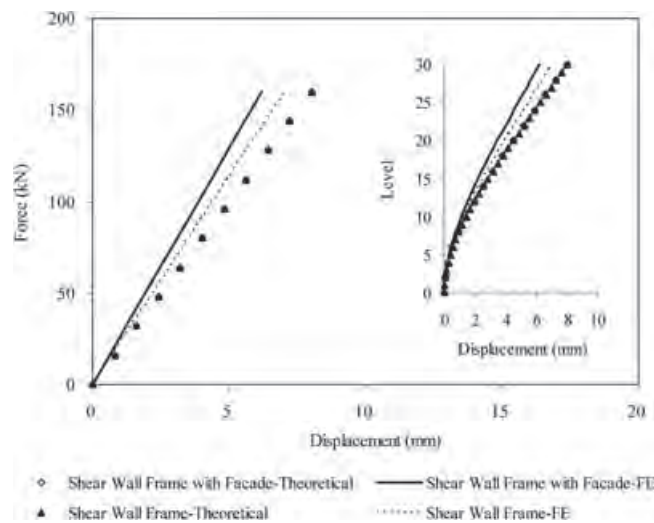


Figure 13. Comparison of the force–displacement relationship and the total storey drift of shear wall frame with and without façade. FE, Finite Element

system analysed in a paper of the authors (Li *et al.*, 2009), it is clear that with the variation of the façade type, the stiffening effect of the façade to the structural system also varies.

3.6.2 Stress distribution within façade panels

Similar to the analysis for the infill walls, maximum allowable deflection (according to the Australian Standards, (AS/NZS 1170 series, 2002)) was applied to the façade panel so that the stress distribution within the panel can be evaluated. Figure 14 shows the maximum tensile stress within the façade panel. It is similar to that of window panels, even though the tensile stress is as high as 1.6 MPa, it is still well within the tensile capacity of glass, which is from 27 MPa to 62 MPa.

4. CONCLUSIONS

This study has analysed the performance of different combinations of structure and NSCs. The stiffness contributions of different components are also identified. Based on the analysis, the following conclusions can be drawn:

- (1) Even though identified as NSCs, infill walls and façade systems are very important in increasing structural stiffness.
- (2) More than a 60% extra stiffness contribution could be made by the infill walls to the lateral load resisting system of the structure based on the structures presented in this paper.
- (3) With the structural frame considered in this paper, if the façade system is a built-in façade, the stiffness of the shear wall frames with façades will be 12% higher than those without façades.
- (4) When considering the serviceability of buildings, windows also have slight contribution (approximately 1.2%) to the structural stiffness of the coupled shear wall even though the stiffening effect is not as significant as that of infill walls.
- (5) The type of façade system influences the overall stiffness of the structure. From a built-in façade (i.e., built-in façade with aluminium mullions) to an off-set façade (Li *et al.*, 2009), the difference of structure stiffness varies by around 0 ~ 16%

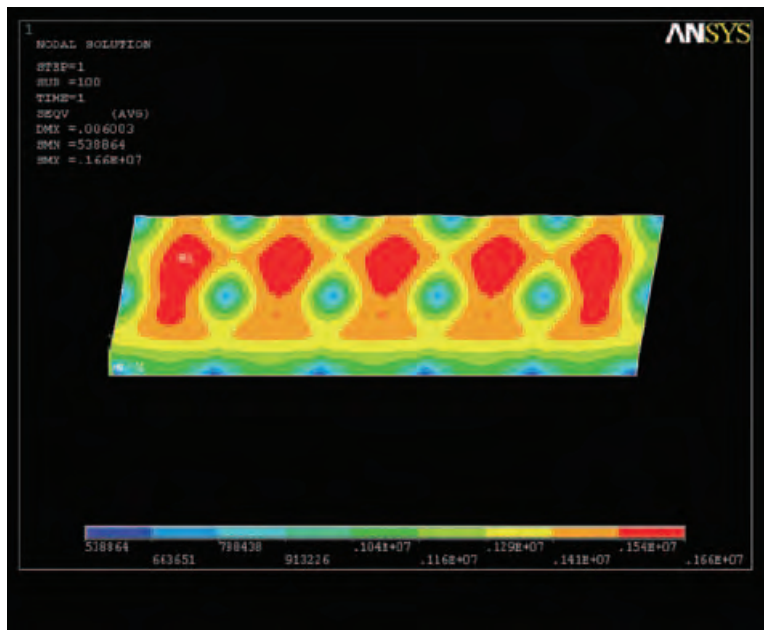


Figure 14. Stress distribution within the façade panel when subject to the maximum allowable storey drift according to Australian Standards (AS/NZS 1170 series 2002) (unit: Pa)

- (6) In the worst scenario, which means direct contact between the infill wall and the structural frame that happens all the time during the maximum service level movement of the building, the concrete block infill walls may not be able to withstand the storey drift under the serviceability limit set by the Australian Standards even though significant stiffness contributions can be realized by including infill walls into the structural analysis.
- (7) Window and façade panels, are capable of adapting to the compliant drift under the serviceability allowances offset by the Australian Standards.

Based on the analyses carried out in this paper, it is necessary that field tests (either model tests or case-study building tests) be conducted to verify and validate the results found using the finite element analysis. Moreover, to further investigate the building performance, simplified but equivalent finite element models should also be developed so that the influences of both individual elements and overall NSCs can be evaluated, to form a basis on which design recommendations could be made.

REFERENCES

- Arnold C. 1991. The seismic response of non-structural elements in buildings. *Bulletin of the New Zealand National Society for Earthquake Engineering* **24**(4): 306–316.
- AS/NZS 1170 series. 2002. *Standards Australia/Standards New Zealand*, ISBN 0 7337 4469 9.
- Freeman SA. 1977. Racking tests of high-rise building partitions. *Journal of the Structural Division, ASCE* **103**(8): 1673–1685.
- Gad EF, Duffield CF, Chandler AM, Stark G. 1998. *Testing of Cold-formed Steel Framed Domestic Structures*, the 11th European Conference on Earthquake Engineering, France, CD Rom.
- Gad EF, Chandler AM, Duffield CF, Stark G. 1999a. Lateral behavior of plasterboard-clad residential steel frames. *Journal of Structural Engineering* **125**(1): 32–39.

THE INFLUENCE OF NON-STRUCTURAL COMPONENTS ON TALL BUILDING STIFFNESS

- Gad EF, Duffield CF, Hutchinson GL, Mansell DS, Stark G. 1999b. Lateral performance of cold-formed steel-framed domestic structures. *Engineering Structures* **21**(1): 83–95.
- Gad EF, Duffield CF. 2000. Lateral behaviour of light framed walls in domestic structures. *Proceedings of the 12th World Conference on Earthquake Engineering*, Auckland, NZ.
- Hall, JF. (Technical Editor) 1995. Nonstructural damage, Northridge earthquake of January 17, 1994 reconnaissance report. *Earthquake Spectra* **1**: 453–513.
- Hoenderkamp JCD, Snijder HH. 2003. Preliminary analysis of high-rise braced frames with facade riggers. *Journal of Structural Engineering* **129**(5): 640–647.
- Hutchinson GL, Collier P, Duffield CF, Gad EF, Li B, Mendis P. 2006. Integration of GPS measurements and secondary structural elements into the serviceability design of structures. In *The Tenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-10)*, Kanak-Nukulchai, W, Munasinghe, S, Anwar, N. (Eds), Bangkok, Thailand, 3–5 August, ISBN 974-8257-25-8, Asian Institute of Technology.
- Li B, Duffield CF, Hutchinson GL. 2007. *Field Investigation Report—Analysis of Connection Properties*, Department of Civil & Environmental Engineering Research Report, RR/Struc/11/2006, The University of Melbourne, Melbourne, VA, Australia.
- Li B, Duffield CF, Hutchinson GL. 2009. The influence of non-structural components on the serviceability performance of high-rise buildings. *Australian Journal of Structural Engineering* **10**(1): 53–62.
- Melchers RE. (Ed) 1990. *Newcastle Earthquake Study*, The Institution of Engineers: Australia.
- McDonnell B. 2001. *What Businesses Learned from the Nisqually Earthquake of February 28, 2001*, The Cascadia Region Earthquake Workgroup (CREW), Seattle, www.crew.org [accessed on 12 Jan 2007].
- Naeim F. 1999. Lessons learned from performance of non-structural components during the January 17, 1994 Northridge Earthquake—case studies of six instrumented multi-story buildings. *Journal of Seismology and Earthquake Engineering* **2**(1): 47–57.
- Phan LT, Taylor AW. 1996. *State of the Art Report on Seismic Design Requirements for Nonstructural Building Components*, US National Institute of Standards and Technology Interim Report.
- Sev A. 2001. Integrating architecture and structural form in tall steel building design. *CTBUH Review* **1**(2): 24–31.
- Stafford Smith B, Coull A. 1991. *Tall Building Structures: Analysis and Design*. New York, NY: Wiley.
- Sugiyama T, Uemura M, Fukutama H, Nakano K, Matsuzaki Y. 2000. Experimental study on the performance of the RC frame infilled cast-in-place non-structural RC walls retrofitted by using carbon fibre sheets. In *Proceedings of the 12th WCEE*, Auckland, New Zealand. (Paper No 2153).

NOMENCLATURE

A	the sum of the areas of right and left walls
A_c	the cross-sectional area of the column
A_d	the cross-sectional area of the brace
GA	the shear rigidity
E	elastic modulus of the material
EI	the stiffness of the coupled shear wall
EI_0	bending stiffness of the outrigger
h_i	the height of storey i
$h_{i-1/2}$	the height from bottom of the structure to the middle of storey i
H_i	the height from bottom of the building to story i
I	the sum of the second moment of areas of individual left and right side walls
I_b	the second moment of area of the coupling beam
I_c, I_g	the second moment of area of column and girder respectively
I_w	the second moment of area of the wall
l	the distance between the centroidal lines of two walls
L	the length of the span
M_c	the restraining moment on the wall due to axial forces in the columns of the facade structure
$M_{i-1/2}$	the moment at the mid-height of storey i
M_0	the moment at the base of the structure

Q_i	lateral force on typical level i
w	the uniformly distributed lateral load
x	the distance measured from the top of the building
y	the lateral deflection of the coupled shear wall at height z
y_{max}	the lateral deflection at the top of the wall
δ_{gi}	the drift caused by girder flexure in typical storey i
δ_{ci}	the drift from column flexure in typical storey i
δ_{if}	the overall drift from bending in typical storey i
δ_s	the shear deflection of braced bents at typical storey i
δ_i	the total drift at storey i
Δ	the top deflection of the structure
λ	the cross-sectional shape factor for shear. It equals to 1.2 in the case of rectangular sections
t	distance between the columns

The influence of non-structural components on the serviceability performance of high-rise buildings *

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SUMMARY: In the last decade there has been an enormous increase in the number of high-rise buildings constructed worldwide. Current Australian high-rise building design practice is to assume that the structural skeleton of a building provides resistance to any lateral forces that might occur. The overall design of high-rise buildings is usually dominated by serviceability limit state considerations rather than the ultimate limit state factors. This study demonstrates the influence of non-structural components (NSCs) (for example, partitions, doors, windows, and façades, etc.) on the lateral stiffness of high-rise buildings, with an emphasis on checks of the design at the serviceability limit state. This paper provides an overview of the design philosophy of high-rise buildings, details of a case-study project, and the results from a detailed parametric study of the case study structure, using a finite element model to identify the influence of the NSCs on the lateral stiffness. Parameters representing NSCs were varied in turn to replicate the reasonable variation that might be found in current Australian practice. Based on the analyses, the influence of NSCs on the lateral stiffness, for example storey drift, was seen to be significant. From a modal analysis, it was observed that there are no significant contributions from the NSCs to the natural frequency of the high-rise building if the NSCs were installed only along the loading direction. However, the second and third modes of the high-rise building increased due to the inclusion of the NSCs. If these components were included in the direction that is orthogonal to the loading direction, obvious changes of the fundamental frequency could be realised by the infill walls.

NOMENCLATURE

A	the sum of the areas of right and left walls	H	the total height of the structure
A_c	sectional area of the column	$h_{v,1/2}$	the height from bottom of the structure to the middle of storey i
A_b	the cross section area of the brace	I	the sum of the second moments of area of individual left and right side walls
$C = \Sigma(I_c)/h_i$	for the columns in storey i of the bent; $\Sigma(I_c)$ is the sum of the second moment of area of the columns in one storey; h_i is the storey height	I_b	the second moment of area of the coupling beam
E	Young's Modulus of the material	I_c & I_g	the second moment of area of the column and girder, respectively
EI_0	bending stiffness of the outrigger	I_w	the second moment of area of the wall
$G = \Sigma(I_g)/L_i$	for the girders across one floor level i of a bent; $\Sigma(I_g)$ is the sum of the second moment of area of the beams in storey i ; L_i is the distance between columns at storey i	l	the distance between the central lines of two walls
GA	the shearing rigidity	M_0	the moment at the base of the structure
		M_c	the restraining moment on the wall due to axial forces in the columns of the façade structure
		$M_{v,1/2}$	the moment at the mid-height of storey i
		Q_i	lateral force on level i
		S	the flexibility parameter for the vertical structure
		S_1	the flexibility parameter for the horizontal structure (outrigger system)

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S_r	the flexibility parameter for the horizontal structure (multi-bay façade system)
w	the uniformly distributed lateral load
x	the distance measured from the top
y_{max}	the lateral deflection at the top of the wall
α	the structure parameter
Δ	the top deflection of the structure
δ_g	the drift caused by girder flexure in storey i
δ_c	the drift caused by column flexure in storey i
δ_b	the overall drift caused by bending in storey i
δ_s	the shear deflection of braced bents at storey i
δ_i	the total drift at storey i
t	distance between the columns
λ	the cross sectional shape factor for shear, which equals 1.2 in the case of rectangular sections
θ_g	the inclination of storey i

1 INTRODUCTION

Various structural forms and materials are employed in the construction of high-rise buildings and lateral loads dominate the real performance of such buildings. The structural response depends on the structural form and materials utilised, and also on the interaction between structural components and non-structural components (NSCs). This paper reports on a study into the merit and effect of integrating the contribution from both the structural skeleton and NSCs on the lateral performance of the building.

1.1 Design development of high-rise buildings

High-rise building design has evolved considerably since the early buildings of the mid-1900s to current structures, such as the Taipei 101 building. The stages of evolution have always sought to make structures stronger, safer, more efficient and more comfortable. Typically, development of high-rise buildings can

be illustrated using the development of high-rise structural forms as depicted in figure 1.

Building height has been constrained by technical aspects such as the structural form and construction materials. When the structure is tall enough, usually more than 40 storeys, the influence from lateral loads such as wind and earthquake frequently dictate the design outcomes when complying with either ultimate limit state or serviceability requirements.

1.2 Current design analysis and limitation

Since World War II, high-rise building design philosophy and relevant international standards have shifted their emphasis from working stress design approaches to limit state design based on generally accepted probability-based approaches. The limit state approach aims to ensure the structure and its constituent components are designed to resist the worst loads and deformations during construction and service, with reasonable safety. Further, the structure must have adequate durability (Stafford Smith & Coull, 1991). The ultimate limit state and serviceability limit state are now used in assessing the performance of the entire structure, or any part of it.

The design process of typical 30 to 50 storey buildings involves designing a skeleton to resist the ultimate limit state loads and the serviceability limit state loads. This process includes allowances for wind and earthquakes. It is common practice for engineers to ignore the structural effects of non-structural elements such as partitions, doors, windows, façades, etc. These NSCs are considered as if they are isolated from the skeleton.

However, buildings are widely recognised as a complex assemblage of both structural skeleton and NSCs (Su et al, 2005). The lateral performance of high-rise buildings is complex because of the conflicting requirements of diverse (structural and non-structural) building systems (Hutchinson et al,

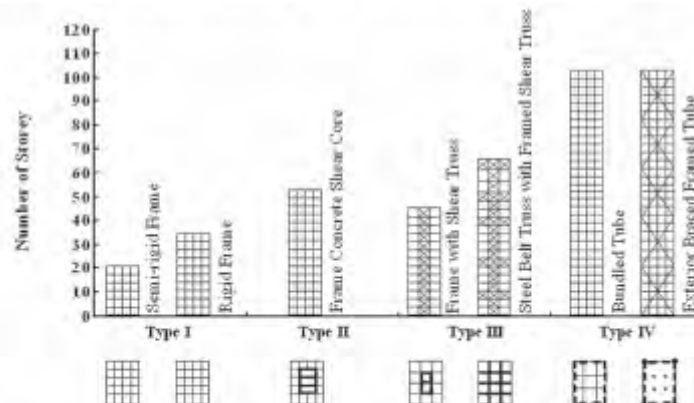


Figure 1: Development of the height and structural form of high-rise buildings.

2006). There is scope to improve the serviceability limit state design requirements over the traditional approach.

NSCs and, in particular, the interaction between structural and non-structural components plays an important role in influencing lateral performance of high-rise buildings (Mahendran & Moor, 1999; Sev, 2001). Because of insufficient consideration of this interaction between different structural systems, the design requirements for internal structural members may become unreasonable (Kuang & Li, 2005). The structural role of NSCs such as façades, infill walls, etc., in resisting the lateral loads is significant and can add to the overall building stiffness by up to 87% (Melchers, 1990; Arnold, 1991; Hall, 1995; Gad et al, 1998; 1999a; 1999b; Naeim, 1999; Hoenderkamp & Snijder, 2003; Su et al, 2005). Thus, the evaluation of the contribution from the NSCs to the overall building performance is necessary. This will provide more detailed information for the design of high-rise buildings.

1.3 Review of the Australian standards

Australian structural engineering standards (eg. AS1170, AS4100, AS3600) detail that: "The design of the structure shall provide load paths to the foundations for forces generated by all types of actions from all parts of the structure, including structural and non-structural components." (Standards Australia, 2002). Moreover, the standards emphasise that "all parts of the structure shall be interconnected". This indicates the importance of integrating the NSCs into the structural system.

However, there are few clear guidelines applicable to NSCs in terms of design or construction practice. No mention is made of any detailed instructions or considerations of the influence of those elements on the overall structural performance.

1.4 Parametric studies

Using the finite element (FE) model described below, parameters representing NSCs were varied in turn to replicate the reasonable variation that might be found in current Australian practice. This enabled the contribution to the lateral stiffness of the building to be assessed.

2 FINITE ELEMENT MODEL DESCRIPTION

A series of FE models were developed to represent different assemblies of elements in a typical tall building. They are:

- the skeleton frame
- frame with service cores
- core-frame with infill walls
- core-frame with shear walls
- core-frame with shear walls and façade.

In the analysis of each model, theoretical verifications are also provided.

The structural frame adopted is symmetric, with the dimension of 30 × 30 × 90 m. The base-to-height ratio of the structure therefore is 1:3. The floor plan is divided into nine bays by columns and beams. The storey height is 3 m (figure 2).

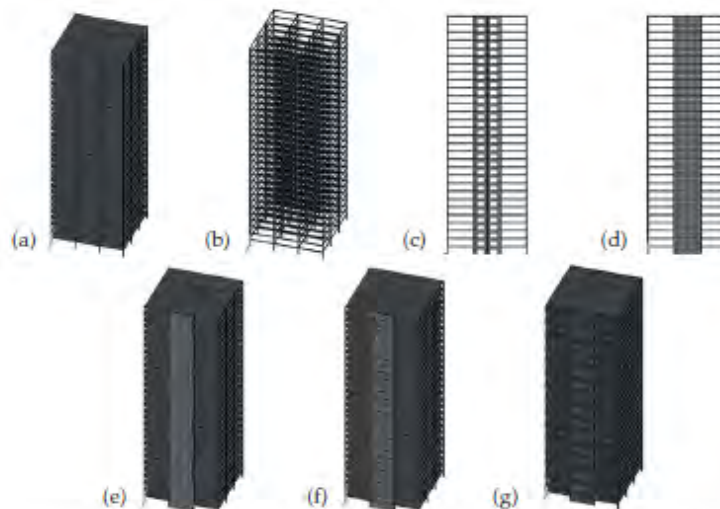


Figure 2: Structural plan – (a) skeleton frame; (b) frame with shear cores; (c) frame with shear cores right view; (d) frame with shear cores front view; (e) core-frame with infill walls; (f) core-frame with shear walls; and (g) core-frame with shear wall façade system.

2.1 Geometric and material properties

To simplify the analysis, a uniformed cross section steel beam 360UB56.7 was assigned to the beams and a steel 310UC118 column to columns (figure 2(a)). The service core is composed of four shear walls: two with openings (coupled shear walls) and two without openings (figures 2(c) and 2(d)). The thickness of these four shear walls is 0.25 m. Concrete block infill walls were included in the two parallel central bays of the frame (figure 2(b)). The gaps between infill walls and the surrounding frame elements are 0.02 m, and mortar bonds are used as contacts between the two parts (figure 3). Two parallel single-bay concrete shear walls were installed on the frame in the shear wall frame structure analysis. Each of the shear walls has the thickness of 0.4 m (figure 2(e)). The façade system is comprised of an aluminium frame and glass panels. The cross-sectional area of the aluminium frame is 0.05×0.05 m. The distance between mullions is 2.5 m. The thickness of the glass panels is 0.02 m. Details of the geometric and material properties of these structural and NSCs are listed in table 1.

2.2 Finite element analysis software

ANSYS10.0 was used in this study to conduct the FE analysis. The advantages of this software are that details of the structure can be well defined, and the non-linearity of the structure and its material

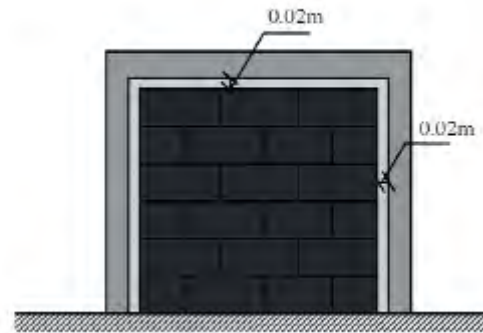







Figure 3: Infill walls.

characteristics or behaviour can also be represented and included. However, there are also disadvantages involving lengthy computational times and larger memory requirements for ANSYS in analysing such a tall building.

2.3 ANSYS elements and boundary conditions

A 3D beam element was used to define the beams and columns of the structural frame, and the frame of the façade system. Shear walls, infill walls and façade glass were represented by 3D shell elements. The possible contact between infill walls and the structural frame was defined by contact pairs that

Table 1: Element and material details.

Element	Dimension	Material		ANSYS element
		Type	Properties	
Column	360UB56.7	Steel	Linear elastic $E = 2.0 \times 10^{11}$ Pa $\mu = 0.29$ density = 7850 kg/m ³	BEAM4
Beam	310UC118			BEAM4
Shear core		Concrete	Linear elastic $E = 2.5 \times 10^{10}$ Pa $\mu = 0.15$ density = 2400 kg/m ³	SHELL63
Shear wall				
Infill wall				
Façade panel		Glass	Viscoelastic $G_0 = 2.74 \times 10^{10}$ Pa $G_v = 6.05 \times 10^{10}$ Pa $1/\beta = 0.53$ density = 2390 kg/m ³	SHELL63
Façade frame		Aluminium	Linear orthotropic $E_x = 3.07 \times 10^{11}$ Pa $E_y = 3.58 \times 10^{11}$ Pa $E_z = 3.58 \times 10^{11}$ Pa $\mu_x = \mu_y = \mu_z = 0.2$ $G_{xy} = G_{yz} = G_{zx} = 1.269 \times 10^{11}$ Pa	BEAM4
Connection		-	k	SPRING14

included both target and contact elements. Details are in table 1.

All six degrees of freedom of the base of the structure were constrained to simulate a fix-ended condition and to eliminate the influence of the foundation. The structural system is symmetric and so is the lateral loading condition. A 0.4 kPa lateral load was incorporated in the model as a series of equivalent point loads applied to the beam/column joints up one side of the model, as shown in figure 4.

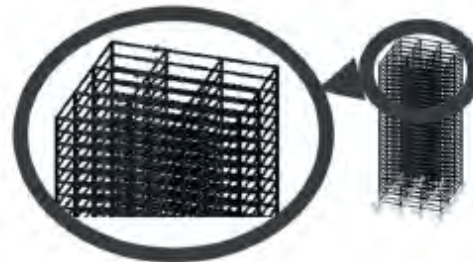


Figure 4: Boundary conditions of the model.

3 ANALYSIS AND DISCUSSION

In this study, both static and dynamic characteristics are investigated, focusing on the differences caused by the NSCs. Storey drifts and natural frequencies are the main foci of the comparison.

3.1 Structural frame

Theoretically, the top deflection of a multi-storey frame can be considered as the accumulation of total storey drifts, which is the sum of storey drifts caused by column flexure, girder flexure and storey drift due to overall bending. This is represented by equations (1) to (6), below (Stafford Smith & Coull, 1991).

3.2 Shear wall-frame

Based on structural theory, the deflection of the wall-frame structure can be calculated by considering that the frame and walls are working together to resist

the lateral loads. Formulae given by Stafford-Smith and Coull (1991) are in equations (7) to (9), below.

3.3 Infilled-frame

Infill walls are always considered as non-structural elements providing bracing effects to the structural frame. Thus, typical theory for infilled-frame structures represents the infill walls by using equivalent bracing elements. The drift in storey i , is a combination of deflection caused by the shear deflection of braced bents at storey i and the total storey drift due to bending (refer to equations (10) to (14)).

$$\delta_s = \frac{Q_i}{2E} \left(\frac{d^3}{L_i^2 A_d} \right) \tag{10}$$

$$\delta_y = h_i \theta_y = h_i A_0^i \tag{11}$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_0 - h_{i-1/2}}{2} \tag{12}$$

$$\delta_s = \frac{Q_i h_i^2}{12E \sum (I_s)_i / L_i} \tag{1}$$

$$\delta_c = \frac{Q_i h_i^2}{12E \sum (I_c)_i / h_i} \tag{2}$$

$$\delta_y = h_i A_0^i \tag{3}$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \tag{4}$$

$$\delta_i = \delta_s + \delta_c + \delta_y = \frac{Q_i h_i^2}{12E \sum (I_s)_i / L_i} + \frac{Q_i h_i^2}{12E \sum (I_c)_i / h_i} + \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \times h_i \tag{5}$$

$$\Delta = \sum \delta_i \tag{6}$$

$$y(z) = \frac{wH^4}{8EI} \left[\frac{8}{(\alpha H)^4} \left[\frac{\alpha H \sinh \alpha H + 1}{\cosh \alpha H} (\cosh \alpha z - 1) - \alpha H \sinh \alpha z + (\alpha H)^2 \left(\frac{z}{H} - \frac{1}{2} \left(\frac{z}{H} \right)^2 \right) \right] \right] \tag{7}$$

$$\alpha^2 = \frac{GA}{EI} \tag{8}$$

$$GA = \frac{12E}{h_i \left(\frac{1}{G} + \frac{1}{C} \right)} = \frac{12E}{h_i \left(\frac{L_s}{\sum (I_s)_i} + \frac{h_i}{\sum (I_c)_i} \right)} \tag{9}$$

$$\delta_i = \delta_s + \delta_f \quad (13)$$

$$\Delta = \sum \delta_i \quad (14)$$

3.4 Shear-wall frame with façade

According to Hoenderkamp & Snijder (2000; 2003), a shear wall structure with a façade system can be considered as an outrigger system. Based on the theory of outrigger systems developed by Stafford-Smith & Coull (1991), equations (15) to (24), below, can be employed to evaluate the performance of multi-storey shear wall frame structures with multi-storey façade system. The connections between the façade system and the structural frame are considered as rigid in this analysis. The influence of the connection properties were discussed in another paper (Li et al, 2007).

Figures 5(a) to 5(e) compare the storey drifts obtained from both the theoretical and FE analysis of the different structural configurations. There is close correlation between the theoretical and FE analysis results for all the models for each configuration.

Figure 6(a) compares the storey drifts of the different assemblies of elements. Under the same lateral loading condition, a reduction of approximately constant 9% in the deflection can be achieved by including the influence of two parallel single-bay multi-storey infill walls to the structural frame (as shown in figure 2). The core-frame structure has a deflection at the top of the building of 22 mm. However, including shear walls in the frame structure reduces the deflection by more than 27%. In taking a further step of including façade panels for the shear wall frame, it can be seen from figure 6(b) that the contribution of façade to the reduction of the structural deflection decreases with the increase of the building height, from around 16% at the bottom to less than 0.1% at the top.

Tables 2 and 3 list natural frequencies of the first three modes of the different models with and without NSCs in the direction orthogonal to the loading direction. In table 2, it is clear that there is no obvious change to the first mode frequency for the skeletal structure with different configurations, since the NSCs were not included in the direction

$$\Delta_b = \frac{wH^4}{8EI} - \frac{1}{2EI} \sum_{n=1}^n M_n (H^2 - x_n^2) \quad (15)$$

$$\begin{bmatrix} M_1 \\ M_2 \\ \vdots \\ M_i \\ \vdots \\ M_n \end{bmatrix} = \frac{w}{6EI} \begin{bmatrix} S_1 + S(H-X_1) & S(H-X_2) & \dots & S(H-X_i) & \dots & S(H-X_n) \\ S(H-X_2) & S_1 + S(H-X_2) & \dots & S(H-X_i) & \dots & S(H-X_n) \\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots \\ S(H-X_i) & S(H-X_i) & \dots & S_1 + S(H-X_i) & \dots & S(H-X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H-X_n) & S(H-X_n) & \dots & S(H-X_n) & \dots & S_1 + S(H-X_n) \end{bmatrix}^{-1} \begin{bmatrix} H^3 - X_1^3 \\ H^3 - X_2^3 \\ \vdots \\ H^3 - X_i^3 \\ \vdots \\ H^3 - X_n^3 \end{bmatrix} \quad (16)$$

$$S = \frac{1}{EI} + \frac{2}{d^2(EA)_c} \quad (17)$$

$$S_1 = \frac{d}{12(EI)_0} \quad (18)$$

$$y_{max} = \frac{wH^4}{8EI_w} - \frac{w(H^3 - x^3)(H^2 - x^2)}{12EI_w} \left\{ \frac{H}{EI_w \{S(H-x) + HS_s\}} \right\} \quad (19)$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \left(\frac{1}{EI_w} + \frac{2}{EA_c \ell^2} \right) (H-x) + \left(\frac{\ell}{12EI_f} + \frac{1}{h_f GA_f} \right) \right\}^{-1} \quad (20)$$

$$\omega = \frac{S_s}{S} = \left\{ \frac{\ell}{EI_f} + \frac{1}{h_f GA_f} \right\} \left\{ \frac{H}{EI_w} + \frac{H}{EA_c \ell^2} \right\}^{-1} \quad (21)$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \frac{H}{S(H-x) + HS_s} \right\} \quad (22)$$

$$S = \frac{H}{EI_w} + \frac{H}{EA_c \ell^2} \quad (23)$$

$$S_s = \frac{\ell}{12EI_f} + \frac{1}{h_f GA_f} \quad (24)$$

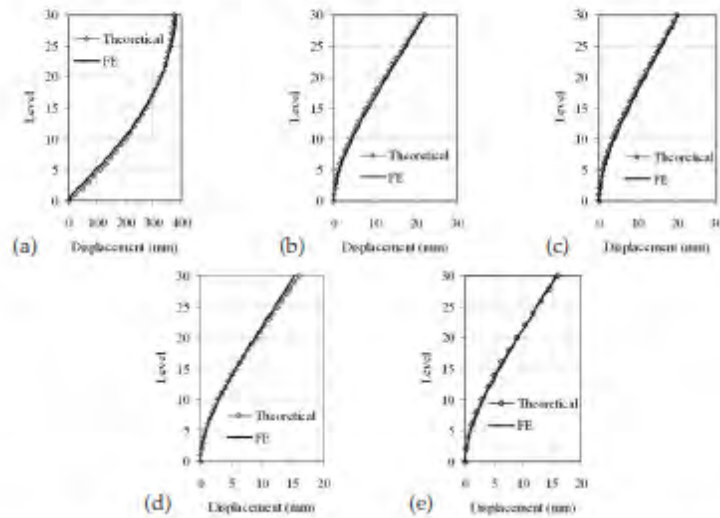


Figure 5: Storey drift of assemblies with different structural and NCSs (lateral loading: 0.4 kPa) – (a) skeleton frame; (b) frame with shear cores; (c) core-frame with infill walls; (d) core-frame with shear walls; and (e) core-frame with shear wall façades.

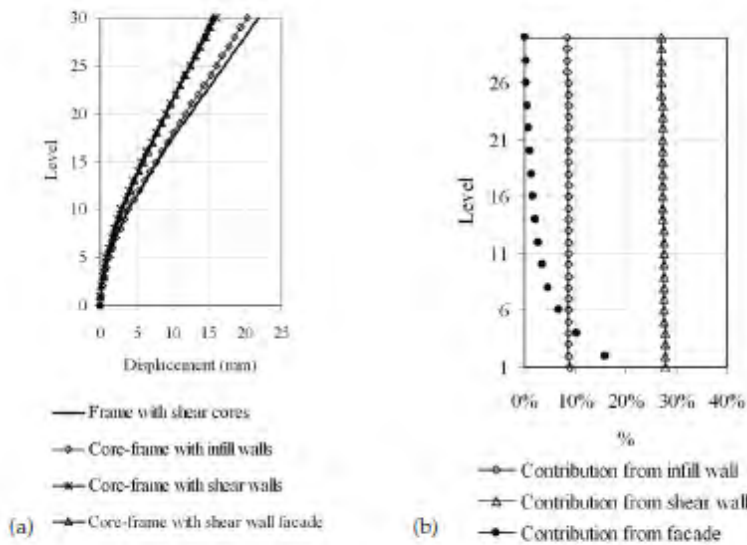


Figure 6: Comparison of the storey drift contributions from different components (lateral loading: 0.4 kPa) – (a) storey drift; and (b) contributions from different components.

orthogonal to the loading direction. However, checking the second and third mode (twisting mode) frequencies, obvious changes appear in different structural configurations. By including shear walls to the core-frame structure, the frequencies of the second and third modes of the structure increased more than 20% and 21%, respectively. Increases of approximately 5% and 9%, respectively, to the second and third modes frequencies are induced by adding infill walls to the structure. This means that if the NSCs are not included in the direction

orthogonal to the loading direction, there will be no significant contribution to the fundamental frequency. However, for the second mode and the twisting mode frequencies, the significant influence of infill walls can be identified while there are minor contributions from façade panels. Table 3 shows how the fundamental frequencies of the structure were changed by including different NSCs orthogonal to the direction of the load. The change caused by adding infill walls to the orthogonal direction of loads is more than 7%, while façade panels only have

Table 2: Frequencies of structures under different modes (without the inclusion of NSCs in the direction orthogonal to the loading direction).

Structural configuration	Mode 1 frequency (Hz)	Mode 2 frequency (Hz)	Mode 3 frequency (Hz)
Frame	0.06	0.07	0.12
Frame + core	0.30	0.32	0.54
Frequency change by core (%)	403.83%	385.97%	364.47%
Frame + core infill wall	0.30	0.34	0.59
Frequency change by infill walls (%)	0.48%	4.93%	9.02%
Frame + core + shear	0.32	0.39	0.65
Frequency change by shear walls (%)	6.54%	20.97%	21.86%
Frame + core shear + façade	0.32	0.39	0.63
Frequency change by façades (%)	-0.13%	-1.38%	-3.52%

Table 3: Frequencies of structures under different modes (without the inclusion of NSCs in the direction orthogonal to the loading direction).

Structural configuration	Mode 1 frequency (Hz)	Mode 2 frequency (Hz)	Mode 3 frequency (Hz)
Frame	0.06	0.07	0.12
Frame + core	0.30	0.32	0.54
Frequency change by core (%)	403.83%	385.97%	364.47%
Frame + core infill wall	0.32	0.34	0.62
Frequency change by infill walls (%)	7.46%	5.99%	15.00%
Frame + core + shear	0.39	0.42	0.70
Frequency change by shear walls (%)	28.57%	29.00%	31.29%
Frame + core shear + façade	0.39	0.42	0.70
Frequency change by façades (%)	0.56%	0.49%	-1.33%

slight influence on the fundamental frequency of the structure (less than 1%).

4 CONCLUSIONS

At present in Australia, it is a common practice for design engineers to ignore the structural effect of non-structural elements in a tall building. NSCs are normally considered as independent and isolated from the skeletal structure.

Based on the analyses reported in this paper, the following conclusions can be drawn regarding the contributions of NSCs to the overall structural lateral performance of the building analysed:

- A noticeable increase in the overall building stiffness occurs by inclusion of the influence of different NSCs to the tall building structure analysed.
- The actual drift of real high-rise buildings is usually less than that predicted by analysis of structural skeleton alone. The inclusion of NSCs, in part, explains the discrepancy.
- The dominant contribution from NSCs appears to be the contribution of infill walls. Given that infill

walls appear to significantly reduce the storey drift, the contribution of block infill stair wells will also reduce the storey drift.

- In real structures, façade panels make a contribution to the stiffness of the overall structure.
- From the modal analysis, it is observed that if NSCs are only included in the parallel direction of the load, there will not be significant contributions of the NSCs to the fundamental frequency of the structure. However, for the second and third modes, significant changes are identified. If NSCs are included both parallel and orthogonal to the loading direction, the study shows that more than 7% increase of the fundamental frequency can be achieved by including infill walls in the structural analysis.
- Given that the NSCs increase the lateral stiffness, these NSCs should be further investigated to ensure their integrity and the robustness during the life of the structure.

In summary, even though the influences of the NSCs on the lateral performance of the high-rise building are not as great as that from the structural components, for example, shear walls, it is worth

paying special attention to the analysis of those NSCs because of their interaction with the skeletal structure.

ACKNOWLEDGEMENT

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REFERENCES

- Arnold, C. 1991, "The Seismic Response of Non-structural Elements in Buildings", *Bulletin of the New Zealand National Society for Earthquake Engineering*, Vol. 24, No. 4, pp. 306-316.
- Gad, E. F., Duffield, C. F., Chandler, A. M. & Stark, G. 1998, "Testing of Cold-formed Steel Framed Domestic Structures", *11th European Conference on Earthquake Engineering*, France, CD Rom.
- Gad, E. F., Chandler, A. M., Duffield, C. F. & Stark, G. 1999a, "Lateral Behavior of Plasterboard-Clad Residential Steel Frames", *Journal of Structural Engineering*, Vol. 125, No. 1, pp. 32-39.
- Gad, E. F., Duffield, C. F., Hutchinson, G. L., Mansell, D. S. & Stark, G. 1999b, "Lateral Performance of Cold-formed Steel-framed Domestic Structures", *Engineering Structures*, Vol. 21, No. 1, pp. 83-95.
- Hall, J. F. T. E. 1995, "Nonstructural Damage, Northridge Earthquake of January 17, 1994 Reconnaissance Report", *Earthquake Spectra*, Vol. 1, pp. 453-513.
- Hoenderkamp, J. C. D. & Snijder, H. H. 2003, "Preliminary Analysis of High-Rise Braced Frames with Facade Riggers", *Journal of Structural Engineering*, Vol. 129, No. 5, pp. 640-647.
- Hutchinson, G. L., Collier, P., Duffield, C. F., Gad, E. F., Li, B. & Mendis, P. 2006, "Integration of GPS Measurements and Secondary Structural Elements into the Serviceability Design of Structures", *The Tenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-10)*, Kanak-Nukulchai, W., Munasinghe, S. & Anwar, N. (editors), Bangkok, Thailand, 3-5 August, ISBN 974-8257-25-8, Asian Institute of Technology.
- Kuang, J. S. & Li, S. 2005, "Interaction-Based Design Formulas for Transfer Beams: Box Foundation Analogy", *Practice Periodical on Structural Design and Construction*, Vol. 10, No. 2, pp. 127-132.
- Li, B., Duffield, C. F. & Hutchinson, G. L. 2007, "Field Investigation Report - Analysis of Connection Properties", Department of Civil & Environmental Engineering Research Report, RR/Struc/11/2006, The University of Melbourne.
- Mahendran, M. & Moor, C. 1999, "Three-Dimensional Modeling of Steel Portal Frame Buildings", *Journal of Structural Engineering*, Vol. 125, No. 8, pp. 870-878.
- Melchers, R. E. (editor), 1990, *Newcastle Earthquake Study*, The Institution of Engineers, Australia.
- Naeim, F. 1999, "Lessons Learned from Performance of Non-structural Components during the January 17, 1994 Northridge Earthquake - Case Studies of Six Instrumented Multi-story Buildings", *Journal of Seismology and Earthquake Engineering*, Vol. 2, No. 1, pp. 47-57.
- Sev, A. 2001, "Integrating Architecture and Structural Form in Tall Steel Building Design", *CTBUH Review*, Vol.1, No. 2, CTBUH.
- Stafford Smith, B. & Coull, A. 1991, *Tall building structures: analysis and design*, Wiley, New York.
- Su, R. K. L., Chandler, A. M., Sheikh, M. N. & Lam, N. T. K. 2005, "Influence of Non-structural Components on Lateral Stiffness of Tall Buildings", *The Structural Design of Tall and Special Buildings*, Vol. 14, No. 2, pp. 143-164.

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QUANTIFICATION OF THE CONTRIBUTION OF NON-STRUCTURAL COMPONENTS TO THE STRUCTURAL PERFORMANCE OF HIGH-RISE BUILDINGS

B. LI¹, C. F. DUFFIELD², and G. L. HUTCHINSON³

ABSTRACT : Non-structural components (NSCs) such as infill walls, façades, stairs, and windows are normally considered as non-load bearing components in the design of buildings. However, a number of researchers have identified that those so-called NSCs have a significant contribution to the lateral performance of the structure. This paper presents the findings of the investigation into the influence of a variety of NSCs on the performance of typical high-rise framed structures via the observation of the influence of these NSCs on the shear and flexural performance, as well as the lateral stiffness of the structures. Finite element (FE) models have been developed to analyse storey drifts, shear force distributions, bending moment distributions, and joint rotations under different structural configurations. The results of the study indicate a significant decrease of the storey drift can be achieved by including different NSCs to the structural frame. Dramatic changes to both the bending moment and the shear force distributions in the inner columns of the structural frame are resulted from the inclusion of NSCs. However, the influence of these NSCs on the flexural and shear performance of the outer columns of the building is significantly less than that of the inner columns. It is concluded that the enhanced performance of actual buildings by including NSCs provides opportunity for refining of the lateral deflection of the building for the serviceability limit states.

KEYWORDS: non-structural components, high-rise building, storey drift, shear and flexural performance

1. INTRODUCTION

Various lateral resisting systems for multi-storey buildings have been developed with the recognition of the importance of the lateral behavior of tall buildings. It is a common practice in Australia that non-structural components (NSCs) are detached from the main structure in the design analysis. However, such isolation will rarely be achieved in the construction process, even if specified. This results in the inevitable transfer of forces from the structural to the non-structural components [1, 2]. A variety of evidence shows that the role of NSCs can be significant in influencing the lateral performance of a structure. The interaction between NSCs and the main structure will lead to loss of serviceability or even occasional failure of the NSCs.

Moment, shear, and the combined capacities are key considerations in design of both individual structural elements and the overall structural system. Changes in the distribution of loads and load paths may cause dramatic variations in the overall structural performance. Thus, substantial changes for both the ultimate limit state design and serviceability limit state design of the structure are possible. According to Taranath [3], the total deflection of the normally proportioned rigid frame can be roughly regarded as a combination of the following four factors:

- Deflection due to the axial deformation of columns (15% ~ 20%)
- Frame racking due to beam rotation (50% ~ 60%)

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- Frame racking due to column rotation (15% ~20%)
- Deflection due to joint deformation (very small)

In this paper, the above influencing factors are re-categorized into two main streams: the flexural performance and the performance under shear forces. The flexural performance of the structure can be expressed by the bending moment distributed along columns and the rotation of the joints. Similarly, the performance under shear forces of the structure can be represented by the shear force distribution in the columns. The findings presented in this study are based on the assumption that the joints of primary structural elements of buildings are rigid.

The objective of this study is to quantify the influence of NSCs on the lateral behavior of the structure, specifically the storey drift and the flexural, shear, and rotational behavior of a typical multi-storey steel-framed structure. Detailed analyses of storey-drift, shear force and bending moment distributions, as well as joint rotations of the structure under different structural configurations have been conducted.

2. ANALYSES PROCEDURES

Analyses have been carried out by developing a series of finite element (FE) models based on a generalized steel-framed structure. The different configurations of the models are: (1) skeleton frame; (2) infill wall frame; (3) shear wall frame; (4) shear wall frame with façade system, refer to Figure 1.

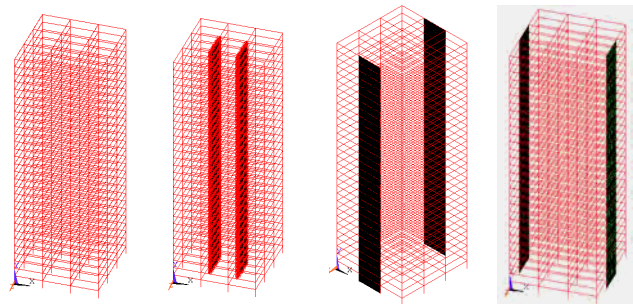


Figure 1. Structural Plan: (a) Structural frame; (b) Frame with infill walls; (c) Frame with shear walls; (d) Shear wall frame with façade system [4]

These different models have been analyzed and discussed for the following scenarios:

- Influence of NSCs on the storey drift of the structure. The results of this analysis have been reported previously, refer to [4].
- Influence of the NSCs on the flexural deflection of the structure. In this analysis, the distributions of the bending moment along the outer and inner columns of the structure have been compared for the four configurations (refer to Figure 1).
- Influence of the NSCs on the shear deflection of the structure. Similar to the analyses of the flexural contributions of NSCs, the shear force distributions of the outer and inner columns of the structure are plotted and discussed.
- Influence of the NSCs on the rotation of the structure. The contribution of the NSCs to the rotational behavior of structures has been quantified.

3. MODEL DESCRIPTION

The models adopted in this study are based on a generalized, highly symmetric, steel-framed structure with the dimensions of 30m by 30m by 90m. The slenderness ratio of the structure therefore is 1:3. The floor plan is divided into 9 bays by columns and beams. The storey height is 3m (Figure 1 (a)). To maintain the symmetric properties of the structure, two parallel shear walls (Figure 1 (b)), and infill walls (Figure 1 (c)) are included along the loading direction. Similarly, two parallel façade panels are installed symmetrically along each shear wall at odd levels (Figure 1 (d)).

In terms of boundary and loading conditions, it is assumed that the base of the structure is fixed for all 6 degrees of freedom. A constant 30kN/m uniformly distributed lateral load is applied to the models.

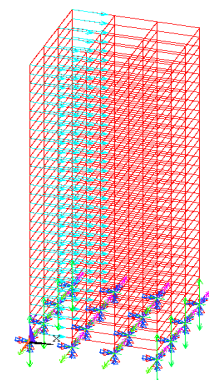


Figure 2. Boundary Condition

Table 1 lists the details of element and material properties.

Table 1. Details of elements and materials [4]

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column	0.4m	Steel	Linear Elastic $E = 2.0 \times 10^{11}$ Pa $\mu = 0.29$ $density = 7850$ kg/m ³	BEAM4
Beam	0.4m			
Shear Wall	0.4m	Concrete	Linear Elastic $E = 2.5 \times 10^{10}$ Pa $\mu = 0.15$ $density = 2400$ kg/m ³	SHELL63
Infill Wall	0.1m			
Façade Panel	0.02m	Glass	Viscoelastic $G_0 = 2.74 \times 10^{10}$ Pa $G_b = 6.05 \times 10^{10}$ Pa $1/\beta = 0.53$ $density = 2390$ kg/m ³	SHELL63
Façade Frame	0.05m	Aluminium	Linear Orthotropic $E_x = 3.07 \times 10^{11}$ Pa $E_y = 3.58 \times 10^{11}$ Pa $E_z = 3.58 \times 10^{11}$ Pa $\mu_{xy} = \mu_{yz} = \mu_{xz} = 0.2$ $G_{xy} = G_{yz} = G_{xy} = 1.269 \times 10^{11}$ Pa	BEAM4
	0.05m			
Connection			k	SPRING14

4. RESULTS AND DISCUSSION

Influence of NSCs on the Inter Storey Drift of the Structure

Full details of the influence of NSCs on the storey drift of the structure were presented in reference [4]. Detailed theoretical computations and finite element analysis were carried out. Close correlation was obtained between the results from the theoretical study and the FE analyses and this confirmed the validity of the model used in this study. A summary of the analyses are shown in Figure 3. It is clear that if the infill walls are installed parallel to the loading direction (as adopted in this study), the storey drift can be reduced by 6%. Similarly, by connecting the façade system to the shear wall frame, the stiffness of the structural system can be improved to 11%.

Influence of NSCs in relation on the Flexural and Shear Performance of the Structure

In this study, the flexural performance and shear performance of the structural frame with different NSCs were analyzed, the bending moment and shear force distributions on both inner and outer columns have been plotted (The specific outer and inner columns are identified in Figure 4 (a) to (d) for each type of the model).

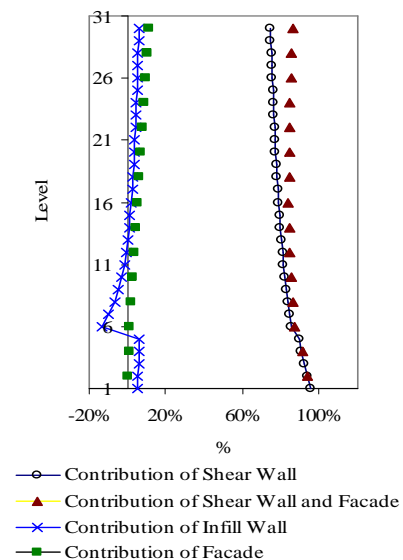


Figure 3. Contributions of Different Components to the Storey Drift of the Building [4]

The deflection of the rigid frame tall building structure is composed of two components: the cantilever bending component and the shear racking component [3]. Taranath also pointed out that bending of the cantilever is mainly caused by the column deformation and it can contribute to 15 ~ 20% of the total deflection of a tall building.

The contribution of infill walls to the bending moment distributions of inner and outer columns of the structure frame is presented in Figure 5. It is clear that under the influence of infill walls, the bending moment along the inner columns increases approximately 10% while at the same time, there is approximately a 2% decrease of the bending moment in the outer columns. This means that adding infill walls to the structural frame will introduce much higher increase of bending moment in the inner columns than that of the outer columns.

The influence of façade panels on the flexural behavior of the structural frame is plotted in Figure 6. It can be observed that façade panels have similar influence upon both the inner and the outer columns of the frame. Approximately 100% contributions can be made by including façade panels to the structural system.

Shear racking of the structure is caused by the deformation of beams and columns. By resisting the shear forces in each floor, the columns bend in double curvature with the contraflexural point being at mid span. The moment at the joints from the columns are resisted by the beams, which also bend in double curvature. This mode of deformation accounts for up to 80% of the total deflection of the structure. [3]

Based on the results from the outer columns, Figure 7, adding infill walls to the structural frame may cause an increase of shear force in the outer columns from ground level to level 28 to maximum of 1.8% while a significant decrease of the shear forces, (up to 7%), occurs from level 28 to level 30.

In Figure 8, the shear forces distributed along the inner columns of the frame with and without infill walls is compared. It is obvious that the infill walls attract huge amount of shear forces to the inner columns of the structural frame, especially at the bottom levels. From Figure 8, the maximum increase of the shear force is at the base of the structure, more than 200kN. That is, the changes of the shear force distribution in the inner columns of the structural frame under the influence of infill walls are significant, from 750% (increase) at the base to -70% (decrease) at the top.

The influence of symmetric façade system on the shear force distribution of both outer and inner columns is plotted in Figure 9. More significant influence on the shear force distributions in the inner columns can be observed. If compared with the main structure without façade panels (shear wall frame), the shear force in the outer columns of the structural frame increased slightly by including

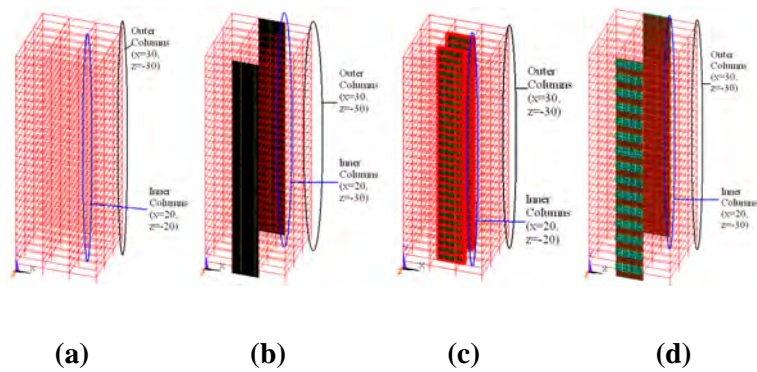


Figure 4. Identification of inner and outer columns in different structural configurations: (a) Structural frame; (b) Frame with infill walls; (c) Frame with shear walls; (d) Shear wall frame with façades

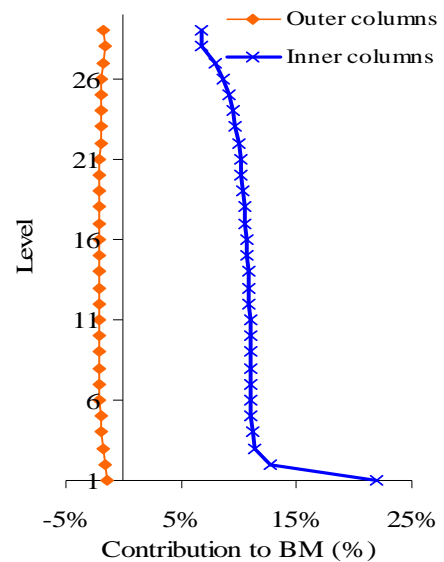


Figure 5. Contributions of Infill Walls to the Bending Moment of the Structure

façade system to the shear wall frame structure, up to 2% at the top level. Meanwhile, the influence of façade panels to the inner column shear force distribution can be as much as 20%.

Influence of NSCs on the Rotation of the Structure

The rotation of the beam and column joint is induced by the deformation of the beam and the column. It has direct relationship with the structure deflection caused by bending moment.

This study analyzed the rotational behaviour of the structural configurations to assess the contribution of NSCs.

The results of the rotations of the joints of the structural frame along the inner and outer columns with different NSCs are presented in Figure 10. It is clear that the rotations along both inner and outer columns of the structural frame are reduced by including infill walls to the structure. Furthermore, the influence of infill walls to the rotation of the inner columns of the structural frame is greater than that of the outer columns. The maximum decreases in the rotations are 0.0005rad (2%) and 0.0012rad (6%) for the outer and inner columns respectively (Figure 11).

The results shown in Figures 10 and 11 also indicate that the contribution of the shear walls in eliminating the frame rotation is significant. More than 90% of the rotations of both inner and outer columns are reduced by adding shear walls to the structural frame.

Regarding the influence of façade panels on the rotational behavior of the shear wall frame, it can be observed that maximum of 6% decrease of the rotation of the joints along the inner columns can be achieved by attaching symmetric façade system to the shear wall frame, while less than 1% of the joint rotation along the outer columns is reduced.

5. CONCLUSIONS

The following conclusions are drawn from this study.

- The storey drift of tall buildings can be significantly reduced by including NSCs in addition to the structural frame when analyzing the structure (note: this is only for serviceability of the structure).
- The influence of NSCs on the flexural performance of tall buildings varies as follows.
 - The influence of infill walls on the bending moment distribution along the inner columns is more significant than that of the outer columns.

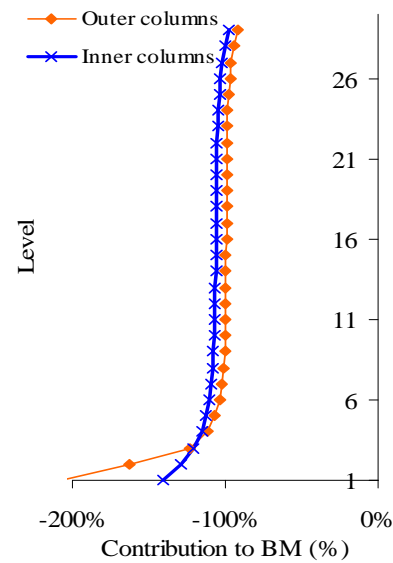


Figure 6. Contributions of Façade Systems to the Bending Moment of the Structure

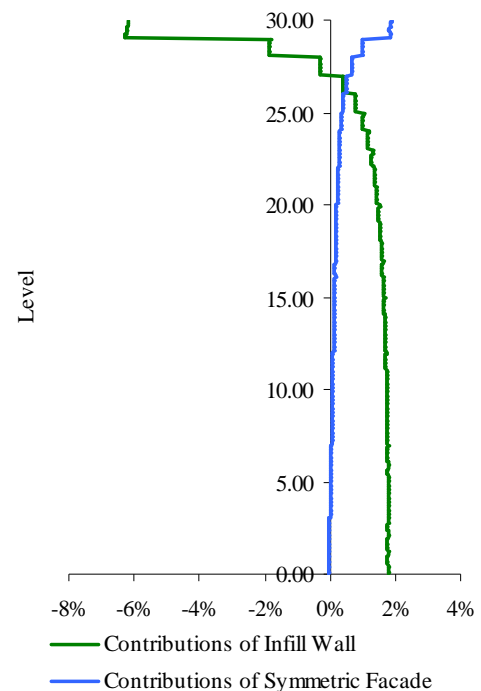


Figure 7. Contributions of NSCs to the Shear Force Distribution of the Outer Columns

Around 10% difference of the bending moment distribution along the inner columns can be induced by attaching infill walls to the frame whilst the contribution of infill walls to the change of bending moment in the outer columns is only 2%.

- The effects of the façade system on the bending moment distributions along both the outer and the inner columns of the shear wall frame are enormous, above 100%.
- A significant influence on the shear force distribution of both inner and outer columns of structural frame can result from including different NSCs to the structural frame. However, similar to the influence on the flexural performance, the influence of NSCs on the shear force distribution in the inner columns is much greater than that of the outer columns.
- The rotational performance of the structural frame is also influenced by the different NSCs. Up to 6% decrease can be realized by including infill walls, while maximum of 8% rotation can be reduced by adding façade panels to the structural frame.
- In comparison, the NSCs have a more significant influence on the bending moment and shear force distributions of the inner columns of the structural frame.
- Even though the influence of NSCs on the flexural and shear performances of the outer columns is not as much as that of the inner columns, it is significant enough to bring the attention to the current design practice.

Some practical applications of the findings are as follows.

When infill walls are included in the analysis of the structure, the flexural and shear capacities of elements around the infill walls need to be carefully analysed and designed owing to the significant changes of the bending moment and shear force distributions caused by the infill walls.

Similarly, if detailed analysis of the shear-wall frame includes façade panels, the moment and shear capacities of the structural elements adjacent to shear walls would require further consideration because of the redistributed bending moment and shear force due to the new (real) load path. This is because the bending moment and shear force diagrams of those elements changed dramatically by adding façade panels to the shear wall frame structure.

The stiffness increase due to the inclusion of the NSCs to the structure may introduce greater tolerance to the serviceability design, i.e. refining the limitations of the inter-storey drift and the total maximum deflection, etc.

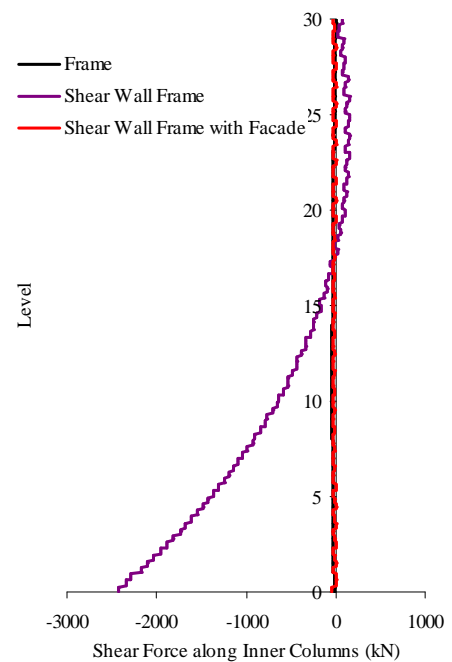


Figure 8. Shear Force Distribution of the Inner Columns with and without Infill Walls

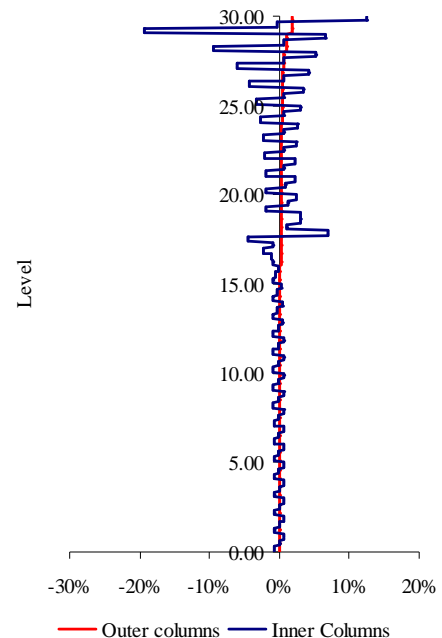


Figure 9. Contributions of Façades to the Shear Force of the Inner Columns

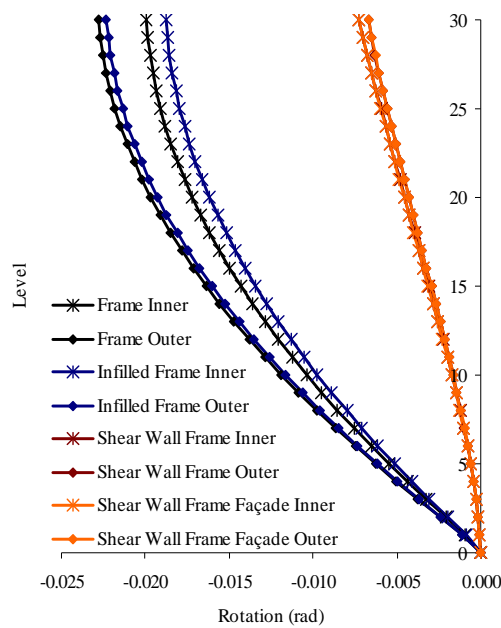


Figure 10. Comparison of the Rotational Behavior of Different Structural Configurations

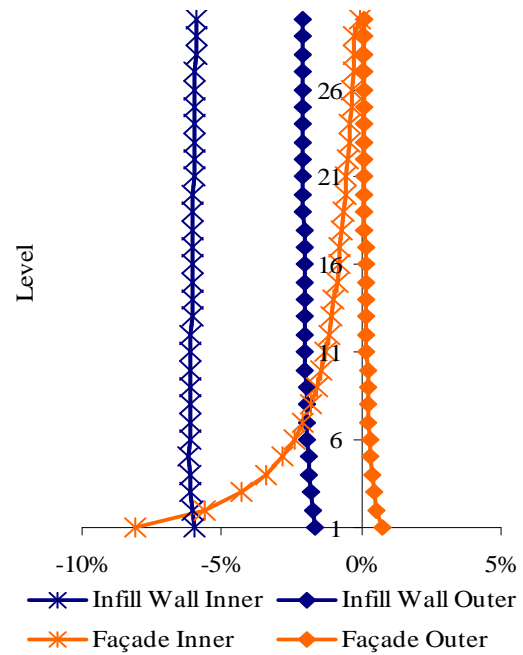


Figure 11. Contributions of NSCs to the Rotational Behaviour of the Structure

REFERENCES

1. Arnold, C., *The seismic response of non-structural elements in buildings*. Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 24, No. 4, 1991, pp 306-316.
2. Freeman, S.A., "Racking tests of high-rise building partitions", *Journal of the Structural Division*, ASCE, Vol. 103, No. 8, 1977, p. pp 1673-1685.
3. Taranath, B.S., *Steel, concrete, and composite design of tall buildings*. 2nd ed., New York: McGraw-Hill. 1998.
4. Li, B, Duffield, C.F, and Hutchinson, G.L., "A Parametric Study of the Lateral Performance of a High-rise Structure". *Proceedings of Australasian Structural Engineering Conference 2008*. 2008. Melbourne, Australia.

Paper No. 057

A Parametric Study of the Lateral Performance of a High-rise Structure

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Abstract

Performance under lateral loads is a significant characteristic of tall buildings, especially when design is for the serviceability limit state. Conventionally, only structural elements such as the frame with beams and columns, shear walls, floor systems, and service core, etc. are considered and designed as the load-bearing components in the strength and stiffness design of tall buildings. However, various studies show that so-called non-structural components (NSCs) such as façades, infill walls, and doors and windows also contribute to the stiffness of the building. This paper investigates the contributions of different structural and non-structural components to the lateral loading performance of a tall steel building by identifying the inter-storey drifts of the structure with different assemblies of elements. Their influence on the dynamic performance of structures is also demonstrated by investigation of the natural frequency of the structure.

A Parametric Study of the Lateral Performance of a High-rise Structure

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Nomenclature:

δ_g	the drift caused by girder flexure in storey i
δ_c	the drift caused by column flexure in storey i
δ_{if}	the overall drift caused by bending in storey i
δ_s	the shear deflection of braced bents at storey i
E	Young's Modulus of the material
I_c, I_g	the second moment of area of the column and girder respectively
Q_i	lateral force on level i .
L	the length of the span,
H_i	the height of story i ,
$M_{i-1/2}$	the moment at the mid-height of storey i
M_0	the moment at the base of the structure
$h_{i-1/2}$	the height from bottom of the structure to the middle of storey i
δ_i	the total drift at storey i , and Δ is the top deflection of the structure.
A_d	the cross section area of the brace
h_i	the height of storey i
Δ	the top deflection of the structure
y_{max}	the lateral deflection at the top of the wall
w	the uniformly distributed lateral load
h, H	the storey height and total height of the structure respectively
I	the sum of the second moments of area of individual left and right side walls
GA	the shearing rigidity
A	the sum of the areas of right and left walls
I_b	the second moment of area of the coupling beam
l	the distance between the central lines of two walls
α	the structure parameter
λ	the cross sectional shape factor for shear which equals 1.2 in the case of rectangular sections
I_w	the second moment of area of the wall
x	the distance measured from the top
A_c	sectional area of the column
t	distance between the columns
EI_0	bending stiffness of the outrigger
M_c	the restraining moment on the wall due to axial forces in the columns of the façade structure

1. Introduction.

The lateral performance of multi-storey buildings is usually considered as being dominated by the skeleton of structural components involving the structural frame, shear walls, concrete cores, etc. However, the increasing evidence has indicated that the role of non-structural components (NSCs), in resisting lateral loads can be very significant. Moreover, the interaction between the NSCs and the structural skeleton may lead to distress, loss of serviceability and occasional failure of the NSCs (e.g. Melchers, 1989; Arnold, 1991; Hall, 1995; Phan, 1996; Naeim, 1999 and McDonnell, 2001). The actual performance of real buildings differs significantly from that of idealised structural

models and studies (Naeim, 1999; Sugiyama et al, 2000). Gad et al (1998, 1999a, 1999b and 2000) have clearly shown that NSCs in low-rise buildings can increase lateral stiffness and strength by more than 100%. This accounts for the observed differences between the theoretical estimate and real performance.

In the current practice, there is not sufficient identification of the structural role played by the NSCs in both Australian and international standards. Most designers of partitions and façades opt for the theoretical approach of complete detachment of these components (i.e. assuming that cladding and partitions do not contribute to the lateral stiffness of the structure). In practice, this would rarely be the case even when gaps are specified. The practicalities of building construction result in the inevitable transfer of forces from NSCs to the skeletal structure and vice versa (Arnold, 1991; Freeman, 1977). This has often resulted in serviceability damage to the NSCs, even after moderate wind or earthquake events.

In order to better understand the role played by the NSCs in influencing structural performance, it is necessary to analyze and evaluate the contribution of each component to the overall lateral performance of multi-storey buildings. This paper parametrically quantifies the contributions from different components of a multi-storey building system to the storey drifts, especially the contributions from NSCs. Effects on the dynamic performance of structures are also demonstrated by the investigation of natural frequencies of the structure.

2. Finite element (FE) model description

A series of finite element models are developed to represent different assemblies of elements of a typical tall building. They are:

- The skeleton frame
- The skeleton frame with infill walls
- The skeleton frame with shear walls
- The skeleton frame with shear walls and façade

In the analysis of each model, theoretical verifications are also provided. The structural frame is a rigid steel frame with a 30m×30m floor plan and main columns supported at every 10m of the span (Figure 1(a)). The total height of the building is 90m, with 3m for each single storey height for 30 storeys. The cross sectional areas of both the steel columns and beams are assumed to be 0.4m×0.4m and the concrete shear walls (Figure 1(c)) and infill walls are assumed to be 0.4m and 0.1m thick (Figure 1(b)), respectively, and are without reinforcement, to simplify the calculation. The façade system (Figure 1(d)) is made by 0.01m thick glass with an aluminium frame which has a cross section area of 0.05m×0.05m. Figure 1 shows the configuration of the different models. Details of material properties are listed in Table 1.

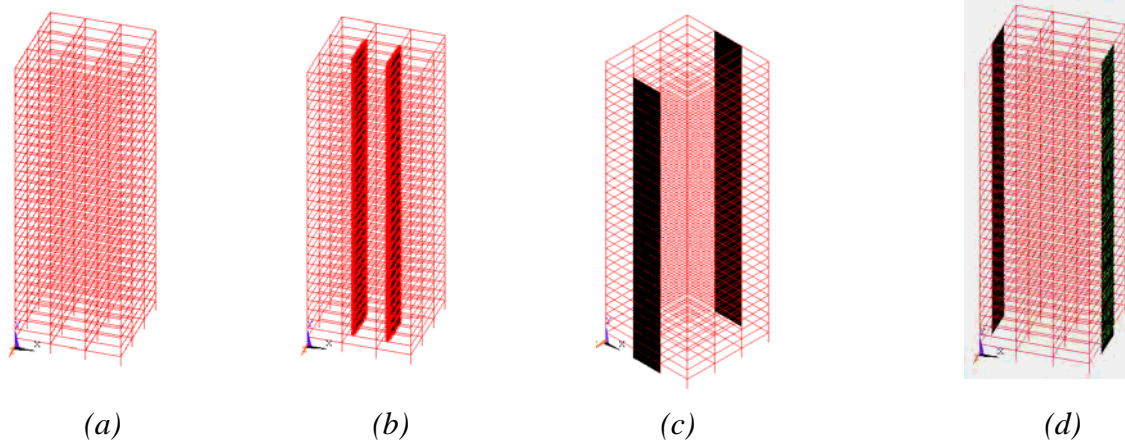


Figure 1. Different Assemblies of the Structure: (a) Frame; (b) Infilled frame; (c) Shear-wall frame; (d) Shear-wall frame with façade;

3. FE analysis software

ANSYS10.0 is used in this study to conduct the FE analysis. The advantages of this software are that details of the structures can be well defined and the non-linearity of the structure and its material characteristics/ behaviour can also be represented and calculated. However, there are also disadvantages with lengthy computational time and the larger memory requirements of ANSYS in analysing such a macro-structures as a tall building.

Table 1. Element and Material Details

Element	Dimension	Material Properties		ANSYS Element
		Type	Properties	
Column		Steel	Linear Elastic $E = 2.0 \times 10^{11}$ Pa $\mu = 0.29$ density = 7850 kg/m ³	BEAM4
Beam				BEAM4
Shear Wall		Concrete	Linear Elastic $E = 2.5 \times 10^{10}$ Pa $\mu = 0.15$ density = 2400 kg/m ³	SHELL63
Infill Wall				SHELL63
Façade Panel		Glass	Viscoelastic $G_0 = 2.74 \times 10^{10}$ Pa $G_b = 6.05 \times 10^{10}$ Pa $1/\beta = 0.53$ density = 2390 kg/m ³	SHELL63
Façade Frame		Aluminium	Linear Orthotropic $E_x = 3.07 \times 10^{11}$ Pa $E_y = 3.58 \times 10^{11}$ Pa $E_z = 3.58 \times 10^{11}$ Pa $\mu_{xy} = \mu_{yz} = \mu_{xz} = 0.2$ $G_{xy} = G_{yz} = G_{yx} = 1.269 \times 10^{11}$ Pa	BEAM4
Connection		--	k	SPRING14

4. Analysis and discussions

In this study, both static and dynamic characteristics are investigated, focusing on the differences caused by the NSCs. Storey drifts and natural frequencies are the main foci of the comparison.

Structural frame. Theoretically, the top deflection of a multi-storey frame can be considered as the accumulation of total storey drift which is the sum of storey drifts caused by column flexure, girder flexure, and storey drift due to overall bending.

$$\delta_g = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L} \right)_i} \quad (1)$$

$$\delta_c = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h} \right)_i} \quad (2)$$

$$\delta_{if} = h_i A_0^i \quad (3)$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \quad (4)$$

$$\delta_i = \delta_g + \delta_c + \delta_{if} = \frac{Q_i h_i^2}{12E \sum \left(\frac{I_g}{L} \right)_i} + \frac{Q_i h_i^2}{12E \sum \left(\frac{I_c}{h} \right)_i} + \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_{i-1/2}}{2} \times h_i \quad (5)$$

$$\Delta = \sum \delta_i \quad (6)$$

Shear wall-frame. Based on the structural theory, the deflection of the wall-frame structure can be calculated by considering that the frame and walls are working together to resist the lateral loads. Formulae given by Stafford-Smith and Coull (1991) are:

$$y(z) = \frac{wH^4}{8EI} \left\{ \frac{8}{(\alpha H)^4} \left[\frac{\alpha H \sinh \alpha H + 1}{\cosh \alpha H} (\cosh \alpha z - 1) - \alpha H \sinh \alpha z + (\alpha H)^2 \left(\frac{z}{H} - \frac{1}{2} \left(\frac{z}{H} \right)^2 \right) \right] \right\} \quad (7)$$

$$\alpha^2 = \frac{GA}{EI} \quad (8)$$

$$GA = \frac{12E}{h \left(\frac{1}{G} + \frac{1}{C} \right)} = \frac{12E}{h \left(\frac{L}{\sum I_g} + \frac{h}{\sum I_c} \right)} \quad (9)$$

Infilled-frame. Infill walls are always considered as non-structural elements with bracing effects to the structural frame. Thus, the typical theory about infilled-frame structures is to represent the infill walls by using equivalent bracing elements. The drift in storey i is a combination of deflection caused by the shear deflection of braced bents at storey i and the total storey drift due to bending.

$$\delta_s = \frac{Q_i}{2E} \left(\frac{d^3}{L^2 A_d} \right) \quad (10)$$

$$\delta_{if} = h_i \theta_{if} = h_i A_0^i \quad (11)$$

$$A_0^i = \frac{M_{i-1/2} + M_0}{EI} \times \frac{h_0 - h_{i-1/2}}{2} \quad (12)$$

$$\delta_i = \delta_s + \delta_{if} \quad (13)$$

$$\Delta = \sum \delta_i \quad (14)$$

Shear-wall frame with façade. According to Hoenderkamp and Snijder (2000, 2003), a shear wall structure with a façade system can be considered as an outrigger system. Based on the theory of outrigger system developed by Stafford-Smith and Coull (1991), the following equations can be employed in evaluating the performance of multi-storey shear wall frame structures with multi-storey façade system. The connections between the façade system and the structural frame are considered as rigid connections in this analysis. The influence of the connection properties will be discussed in another paper.

$$\Delta_0 = \frac{wH^4}{8EI} - \frac{1}{2EI} \sum_{i=1}^n M_i (H^2 - x_i^2) \quad (15)$$

$$\begin{bmatrix} M_1 \\ M_2 \\ \vdots \\ M_i \\ \vdots \\ M_n \end{bmatrix} = \frac{w}{6EI} \begin{bmatrix} S_1 + S(H - X_1) & S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ S(H - X_2) & S_1 + S(H - X_2) & \cdots & S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_i) & S(H - X_i) & \cdots & S_1 + S(H - X_i) & \cdots & S(H - X_n) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ S(H - X_n) & S(H - X_n) & \cdots & S(H - X_n) & \cdots & S_1 + S(H - X_n) \end{bmatrix}^{-1} \begin{bmatrix} H^3 - X_1^3 \\ H^3 - X_2^3 \\ \vdots \\ H^3 - X_i^3 \\ \vdots \\ H^3 - X_n^3 \end{bmatrix} \quad (16)$$

$$S = \frac{1}{EI} + \frac{2}{d^2(EA)_c} \quad (17)$$

$$S_1 = \frac{d}{12(EI)_0} \quad (18)$$

$$y_{\max} = \frac{wH^4}{8EI_w} - \frac{w(H^3 - x^3)(H^2 - x^2)}{12EI_w} \left\{ \frac{H}{EI_w \{S(H - x) + HS_\tau\}} \right\} \quad (19)$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \left(\frac{1}{EI_w} + \frac{2}{EA_c \ell^2} \right) (H - x) + \left(\frac{\ell}{12EI_f} + \frac{1}{hGA} \right) \right\}^{-1} \quad (20)$$

$$\omega = \frac{S_r}{S} = \left\{ \frac{\ell}{EI_r} + \frac{1}{hGA_r} \right\} \left\{ \frac{H}{EI_w} + \frac{H}{EAc^2} \right\}^{-1} \quad (21)$$

$$M_c = \frac{w(H^3 - x^3)}{6EI_w} \left\{ \frac{H}{S(H - x) + HS_r} \right\} \quad (22)$$

$$S = \frac{H}{EI_w} + \frac{H}{EAc^2} \quad (23)$$

$$S_r = \frac{\ell}{12EI_r} + \frac{1}{hGA_r} \quad (24)$$

Figures 2 (a) to (d) compare the storey drifts obtained from both the theoretical and FE analysis of the different structural configurations. There is close correlation between the theoretical and FE results. The maximum difference between these two types of results appears in the analysis of the shear-wall frame with a façade system, and is around 50%.

Figure 3(a) compares the storey drifts of the different assemblies of elements. When under a uniformly distributed load of 30 kN/m, around 5mm reduction of the deflection can be achieved by adding only two parallel single-bay multi-storey infill walls to the structural frame (as shown in Fig. 1). Regarding the shear-wall frame structure, a significant decrease of the top deflection of 54 mm can be obtained by combining shear walls with structural frame. This compares with the frame structure, which has the top deflection of 78 mm. In taking a further step of including façade panels for the shear wall frame, it can be seen from Fig. 3(a), that another 10mm reduction of top

deflection can be realized. Figure 3(b) summarizes the influences of the different NSCs on the building storey drift under a 30kN/m uniformly distributed lateral load. It is found that the maximum contributions from façades and infill walls can be as much as 19% and 6% respectively.

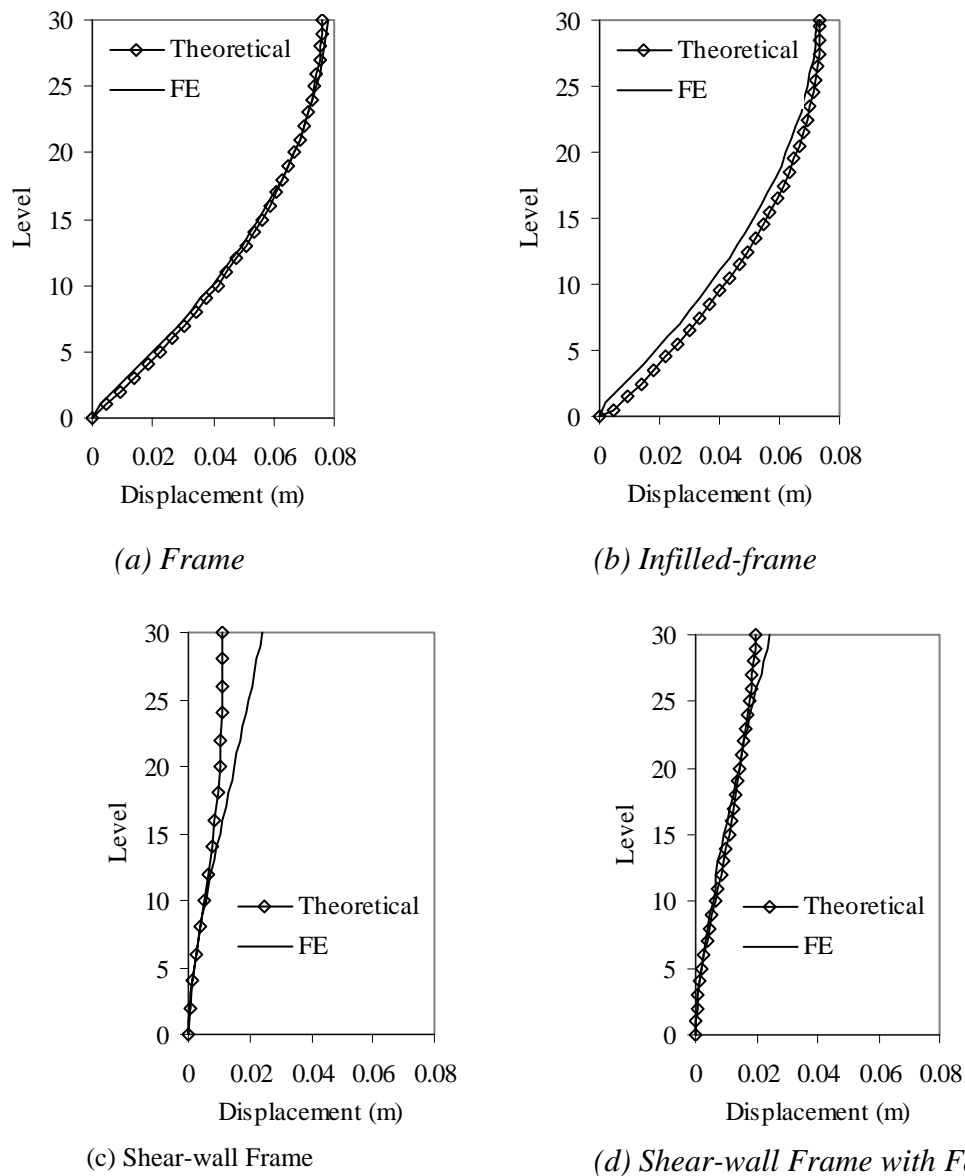


Figure 2. Storey drift of Assemblies with Different Structural and Non-structural Components
 (Lateral loading: 30kN/m)

Considering the dynamic characteristics, Table 2 lists natural frequencies of the first 5 modes of the different models. Significant changes of the first and second modes frequencies (5.6% and 78.1%, respectively) can be achieved by adding the shear walls to the structural frame. However, only 0.006Hz (1.9%) difference of the frequencies is induced by the infill walls, whilst almost zero contribution from the façade panels. This means that the NSCs do not have a large contribution to the dynamic performance of a tall building.

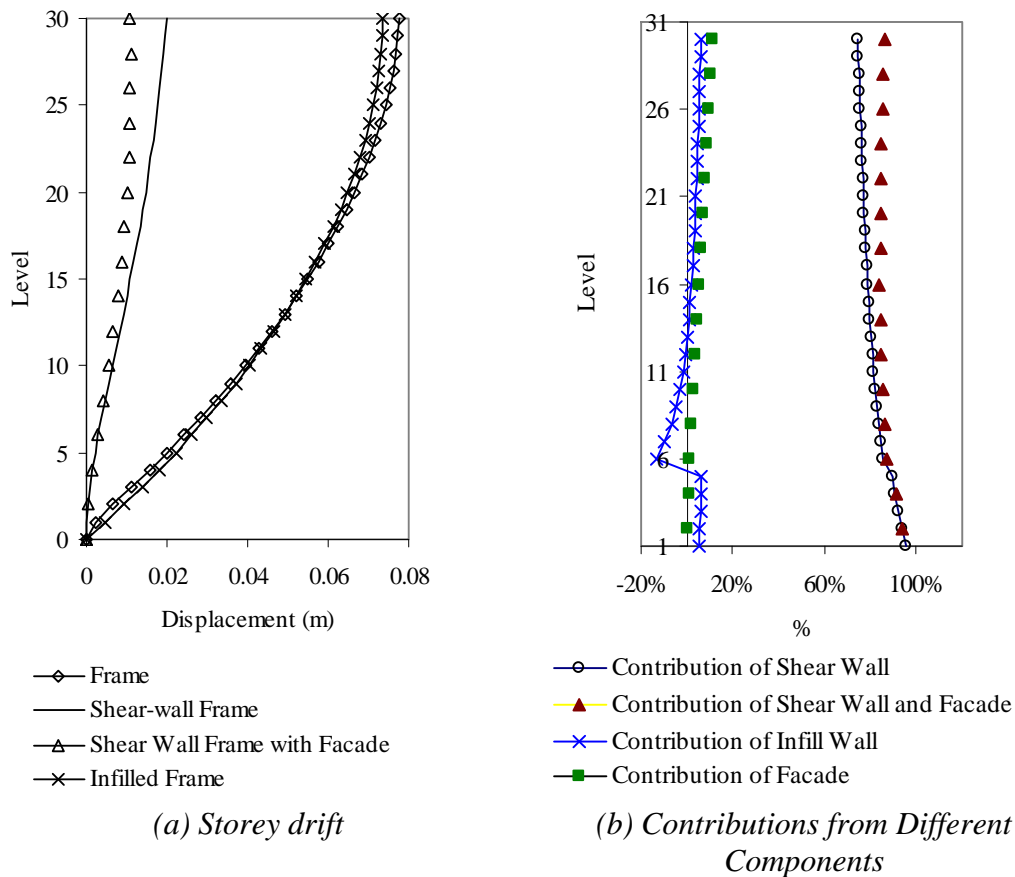


Figure 3. Comparison of the Storey Drift Contributions from Different Components (Lateral loading: 30kN/m)

Table 2. Frequencies of structures under different modes

Structure Configuration	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
Frame	0.319	0.319	0.367	0.963	0.963
Infilled Frame	0.313	0.322	0.406	0.944	0.970
Shear-wall Frame	0.301	0.568	0.624	0.906	1.531
Shear-wall Frame with Façade	0.301	0.569	0.625	0.906	1.530

5. Conclusions

Based on the above observations, following conclusions can be drawn regarding the contributions of NSCs to the overall structure lateral performance of the building analyzed:

- a noticeable increase of stiffness can be realized by adding different NSCs to the tall building structure analyzed;
- by combining only two parallel single-bay, multi-storey infill walls to the structural frame, the storey drift of the structure can be reduced by up to 5mm (approximately 6%);
- façade panels can also add stiffness to the structure. More than 11% of the improvement of the deflection control can be achieved by attaching single-bay façade panels to only even or odd storeys of the shear-wall frame structure;
- From the modal analysis, it is observed that there are no significant contributions from the NSCs (only 1.88%) to the dynamic performance of the tall building.

In summary, even though the influences of the NSCs on the lateral performance of this tall building are not as great as that from the structural components, it is worth paying special attention to the

analysis of those NSCs because of their significant contribution particularly to the static characteristics of the structure utilized in this study.

6. References

- Arnold C. The seismic response of non-structural elements in buildings. *Bulletin of the New Zealand National Society for Earthquake Engineering* 1991; 24(4), pp 306-316.
- Freeman, S.A. Racking tests of high-rise building partitions. *Journal of the Structural Division* 1977; 103(8), pp 1673-1685.
- Gad, E.F., Duffield, C.F. Interaction between brick veneer walls and domestic framed structures when subjected to earthquakes. *Proceedings of the 15th Australasian Conference on Mechanics of Structures and Materials, Melbourne; 1997.*
- Gad, E.F., Chandler, A.M, Duffield, C.F. Stark, G. Testing of cold-formed steel framed domestic structures. *Proceedings of the 11th European Conference on Earthquake Engineering France;1998.*
- Gad, E.F., Duffield, C.F., Hutchinson, G.L., Mansell, D.S., Stark, G. Lateral performance of cold-formed steel-framed domestic structures. *Engineering Structures* 1999a; 21(1), pp 83-95.
- Gad, E.F., Chandler, A.M., Duffield, C.F., Stark, G. Lateral Behaviour of plasterboard-clad residential steel frames. *Journal of Structural Engineering* 1999b; 125(1), pp 32-39.
- Gad, E.F., Duffield, C.F. Lateral behaviour of light framed walls in domestic structures. *Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, NZ; 2000.*
- Gad, E.F, Lam, N.T.K., Duffield, C.F., Hira, A., Chandler, A.M. Seismic behaviour of non-structural components in high-rise buildings. *Proceedings of the 12th European Conference on Earthquake Engineering, London; 2002.*
- Hall, J.F. Nonstructural damage, Northridge earthquake of January 17, 1994 reconnaissance report. *Earthquake Spectra* 1995; Vol.1 Publication 95-03, pp 453-513.
- Hoenderkamp, J. C. D., Snijder, H. H. Simplified analysis of facade rigger braced high-rise structures. *Structural Design of Tall Buildings* 2000; 9(4): 309-319.
- Hoenderkamp, J. C. D., Snijder, H. H. Preliminary analysis of high-rise braced frames with facade riggers. *Journal of Structural Engineering* 2003; 129(5): 640-647.
- McDonnell B. What businesses learned from the Nisqually earthquake of February 28, 2001. *The Cascadia Region Earthquake Workgroup (CREW), Seattle (www.crew.org); 2001.*
- Melchers, R.E. (Editor) *Newcastle Earthquake Study*. The Institution of Engineers, Australia; 1990.
- Naeim F. Lessons learned from performance of non-structural components during the January 17, 1994 Northridge Earthquake – case studies of six instrumented multistory buildings. *Journal of Seismology and Earthquake Engineering* 1999; 2(1), pp 47-57.
- Phan L.T., Taylor A.W. State of the art report on seismic design requirements for nonstructural building components. In: *US National Institute of Standards and Technology Interim Report (NISTR) 1996, 5857.*
- Stafford-Smith, B., Coull, A. *Tall Building Structures: Analysis and Design*. New York: Wiley; 1991.
- Sugiyama T., Uemura, M., Fukutama, H., Nakano, K. Matsuzaki, Y. Experimental study on the performance of the RC frame infilled cast-in-place non-structural RC walls retrofitted by using carbon fibre sheets, *Proceedings of the 12th WCEE, Auckland, New Zealand; 2000.*

Simplified Finite Element Modelling of Multi-storey Buildings: The Use of Equivalent Cubes

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ABSTRACT: Finite element modelling is frequently used to overcome experimental limitations in predicting and analysing the performance of structures. However, constrained by software restrictions, 3-D analysis of high-rise buildings is still challenging and complex. This paper discusses how to substructure different parts of a multi-storey building with cubes having equivalent stiffness properties. As a result, the mesh density of the whole building is reduced significantly and the computational time and memory normally consumed by such complex structural dimensions and material properties will also be reduced. The simplified analysis results of a high-rise frame structure with a concrete core have been used to explore the reliability of this method.

KEYWORDS: Multi-story Buildings, Equivalent cubes

1 INTRODUCTION

Finite element modelling is frequently used to overcome experimental limitations in predicting and analysing the performance of structures. In designing and analyzing the performance of high-rise buildings, it is especially important that an effective modelling technique be involved because of the complexity of the real structural behavior and the difficulties of full scale measurement.

To date, various modelling methods have been developed to analyse the performance of high-rise buildings [1-6]. The "Finite Story Method" introduced by Pekau et al. [1, 2] can reduce the unknowns of each storey in a high-rise building thus improving greatly the computing efficiency. The program developed by Ozturun et al. [3] has a special mesh generation subroutine and graphics program for the finite element analysis of shear walls in buildings. Beams or columns can be easily added or deleted in this program, which makes the modelling process more convenient. Mahendran et al. [4] believed that 2-D modelling analysis is not sufficient to predict the real performance of structures, so a 3-D modelling method for steel portal frame buildings is necessary. Poulsen et al. [5] gave details of how to consider the reinforcing bars and the tension/compression behaviour of concrete in the limit state analysis of reinforced concrete plates subjected to in-plane forces. This is especially useful for the analysis of single reinforced elements. When modelling high-rise structures, where there are often concerns about node limitations and growing computational time

and memory capacity of finite element analysis tools such as ANSYS, this method might be appropriately used in the substructure. A super-element method introduced by Kim et al [6] for modelling shear wall structures is a method involving substructures. This method can easily achieve equal accuracy within reduced computing time.

It is also found that a great deal of modelling work has focused on the seismic or wind behaviour of structures [7-16] since these two types of lateral loads are the most serious external loads which may cause severe damage to high-rise buildings. Almost all of these models are about limit state analysis or prediction. People can now be confident about the seismic or wind analysis of framed [7, 14] and reinforced concrete shear wall structures [8] because of research within above area. However, most of these methods are based on 2-D models which involve a lot of simplifications compared to the real performance of a 3-D structure. Even though some 3-D models were used in the analyses, those models were limited to modelling single elements. It appears that, the above situation is due largely to the limitations of current FE analysis tools. As pointed by Ozturun et al. [3], due to the large and complex amount of input requirements and node limitations, the utilization of some other finite element analyzing software such as SAP90, etc. seems impractical.

Constrained by software restrictions, 3-D analysis of high-rise buildings is a big challenge, especially when analyses of the contributions of non-structural components to the building stiffness are required. To focus on the interaction details between structural

and non-structural components, a simple but efficient primary structural model needs to be developed first.

This study concerns the development of a simple primary structural model. A method called “The Equivalent Cubic Method” is presented together with a calibration analysis of the Force-Displacement (F-D) relationship under static loading conditions.

2 STRUCTURAL MODEL

The proposed structure is a 32-storey high-rise reinforced concrete building. The height of each storey is 3m, and the floor plan is composed of a concrete core and rigid frame as shown in Figure 1.

To simplify the modelling and analysis procedure, this floor plan has been divided into series of sets of 9 blocks, which can be categorized into 3 different types according to dimensions and properties of their structural elements (Figure 2).

Area type I is the 15×15 m concrete core block. It includes a set of 0.4m-thick shear walls, 4 head beams of shear walls with cross section area of 0.6×0.6 m, and 4 columns of 0.8×0.8 m cross section area standing at the 4 corners of the core (Figure 2).

Area type II refers to the four corner parts of the frame (Figure 2). Within those 45×45 m areas, orthogonal beams divide each area into 9 sections of 15×15 m (Figure 2).

Area type III involves the four 45×15 m rectangular areas which have common walls with the core area. Similar to type II, the rectangular floor slab is supported by 3 beams along its longer span (Figure 2). Details of each element are provided in Table 1.

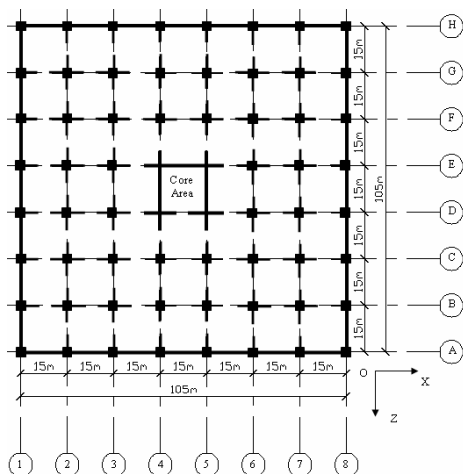


Figure 1. Typical Floor Plan of the 32-Storey High-rise Building

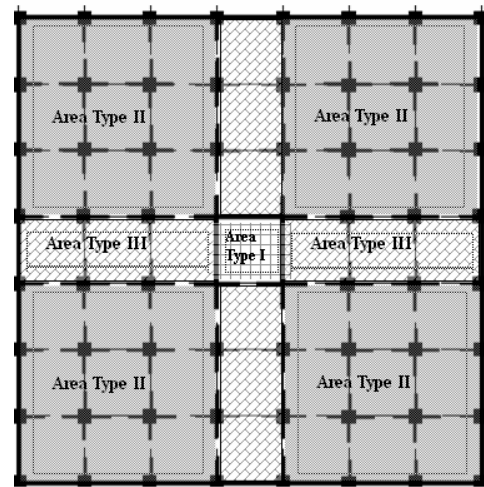


Figure 2. Divide the Floor Plan into 9 Blocks According to the Dimension

3 EQUIVALENT MODEL

In this study, the commercial software package ANSYS 10.0 has been used as the analytical tool. The largest constraint in this structural model is that the computational capability of ANSYS will be influenced by both the computer hardware and the meshing density. The challenge for this simulation process is to save both computing time and memory by efficiently reducing the overall meshing density of the structure.

The aim of this study is to find an efficient equivalent model to represent the real structural model for the serviceability analysis of high-rise buildings. Some details such as connection properties, etc. can be simplified. And, when designing the models, following assumptions have been made:

Ignore openings in the structure;

The material used is pure concrete without reinforcement;

All structural components (beams, columns, walls, and floor slabs) are considered have rigid connections to each other;

The procedure for the model simplification is:

- Structural model. Create a one-storey concrete core model of the structure (Type I) according to the component details and material properties given in previous section. The mesh elements used by ANSYS have been listed in Table 2;
- Static analysis 1. Process static analysis of this core block. Plot the Force-Displacement (F-D) relationship of the top edge point of the block.
- Cubic model. Build a 3×3×3 m cubic model, with the 4 side-faces as walls, and the top and bottom as floor slabs, and the linear joints as beams and columns respectively. The mesh elements used by ANSYS have been listed in Table 2.

Table 1. Details of Structural Components

Area Type	Structural Component	Area (m ²)	Thickness (m)	I _{zz} (m ⁴)	I _{yy} (m ⁴)	Re-bar Diameter (mm)	Concrete grade (Mpa)
Type I	Beam	0.36		0.0108	0.0108	N16~N36	32
	Column	0.64		0.0341	0.0341	N16~N36	80~32
	Wall		0.4			N16~N36	
	Floor slab		0.2			N12~N36	32~40
Type II	Beam I	0.36		0.0108	0.0108	N16~N36	32
	Beam II	0.16		0.0021	0.0021	N16~N36	32
	Column	0.64		0.0108	0.0108	N16~N36	80~32
	Internal Wall		0.2			N16~N36	
Type III	External Wall		0.4			N16~N36	
	Floor slab		0.2			N12~N36	32~40
	Beam I	0.36		0.0108	0.0108	N16~N36	32
	Beam II	0.16		0.0021	0.0021	N16~N36	32
Type III	Column	0.64		0.0108	0.0108	N16~N36	80~32
	Internal Wall		0.2			N16~N36	
	External Wall		0.4			N16~N36	
Type III	Floor slab		0.2			N12~N36	32~40

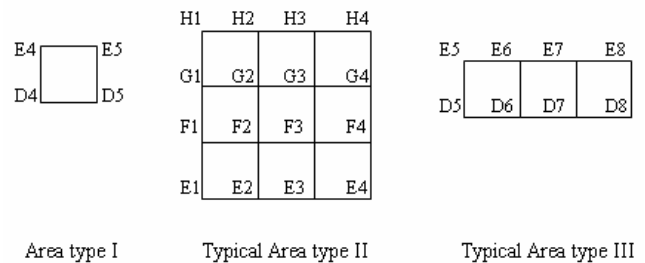
- Static analysis 2 & F-D relation calibration. Repeat the static analysis in step 2 on the cubic model. Use the F-D relationships achieved from both step 2 and step 4 in calibrating.
- Equivalent cubic model. Finally, adjust the properties of structural components and get the equivalent cubic model of the one-storey concrete core block from the calibration process in step 4.
- Other Type of Area of Structure. Repeat the above step 1-5 to get the equivalent cubic models of block types II and III (all the cubic models should be 3×3×3 m because of the geometric considerations).

Relevant concrete material properties and modeling elements used throughout the building are detailed in Table 2. Figure 3 presents a representation of boundary gridlines with cubic areas and Table 3 details boundary constraints for each area.

Table 2. Meshing Elements Used in ANSYS10.0

	BEAM 4	SHELL6 3	Concrete Property
Beam	√		
Column	√		
Wall		√	
Floor Slab		√	

Figure 3. Boundary Definition of Different Area Type



4 RESULT OF CALIBRATION

Calibration of F-D relations of structural models and cubic models has been presented in Figure 4. In the static analysis, a concentrated external load $F=2 \times 10^8$ N has been applied to the top middle point of each model. From Figure 4, the maximum top corner displacements of the structural core model and the equivalent cubic model are 26.8mm and 27.36mm respectively, i.e. the deviation is only 2.11% (Table 4). The F-D relations of structural models and the equivalent cubic models calibrate with each other perfectly. It is observed that when subject to external static loads, the equivalent cubic model for each part of the structure has almost the same behaviour as the relevant part of real structure.

The calibration of the F-D relationships that may occur to cubic model in an asymmetric condition when under the lateral concentrated loads is plotted in Figure 5. From the results, the maximum difference from that calibration is only 1.78% (Table 5). It is found that similar to the symmetric model, the

equivalent cubic model can perform in exactly the same way as the real structure in both directions.

Table 3. Boundary Conditions of the Model

		UX	UY	UZ	ROTX	ROTY	ROTZ	Reference Frame
Typical Area Type I	D4-E4	--	0	--	--	--	--	
	E4-E5	--	0	--	--	--	--	
	E5-D5	0	0	0	0	0	0	
	D5-D4	--	0	--	--	--	--	
Typical Area Type II	E1-F1-G1-H1	--	0	--	--	--	--	
	H1-H2-H3-H4	--	0	--	--	--	--	
	H4-G4-F4-E4	0	0	0	0	0	0	
	E4-E3-E2-E1	--	0	--	--	--	--	
	F1-F2-F3-F4	--	0	--	--	--	--	
	G1-G2-G3-G4	--	0	--	--	--	--	
	E2-F2-G2-H2	--	0	--	--	--	--	
Typical Area Type III	E3-F3-G3-H3	--	0	--	--	--	--	
	D5-E5	--	0	--	--	--	--	
	E5-E6-E7-E8	0	0	0	0	0	0	
	E8-D8	--	0	--	--	--	--	
	D5-D6-D7-D8	--	0	--	--	--	--	
	D6-E6	--	0	--	--	--	--	
	D7-E7	--	0	--	--	--	--	

Under surface loads such as pressure, because of the difference in geometrical dimensions, the results are not so close. Similarly, owing to the spatial difference, and inequality of density distribution, distinct differences exist in the modal shapes of the two types of models.

The calibration results under different loading conditions show that this simplified method of modelling high-rise structures is suitable in static analysis for structural serviceability. It can simulate the exact F-D performance of a structure and thus can effectively save computational time and memory. Moreover, there are two other main advantages in using this simplified model to analyse the behaviour of a high-rise building.

This “Equivalent Cubic Method” can be conveniently used in modelling different buildings. The stiffness calibration between the structural model and the cubic model can be readily conducted no matter what kind of floor plan, element properties, or material properties need to be involved. Furthermore, asymmetric structures can really be modelled by this cubic approach.

A further benefit of this simplified model is the convenience it would bring to the analysis of the influence of different non-structural components to high-rise building performance. Non-structural com-

ponents can easily be modelled using shell or spring elements and connected to the main structural part,

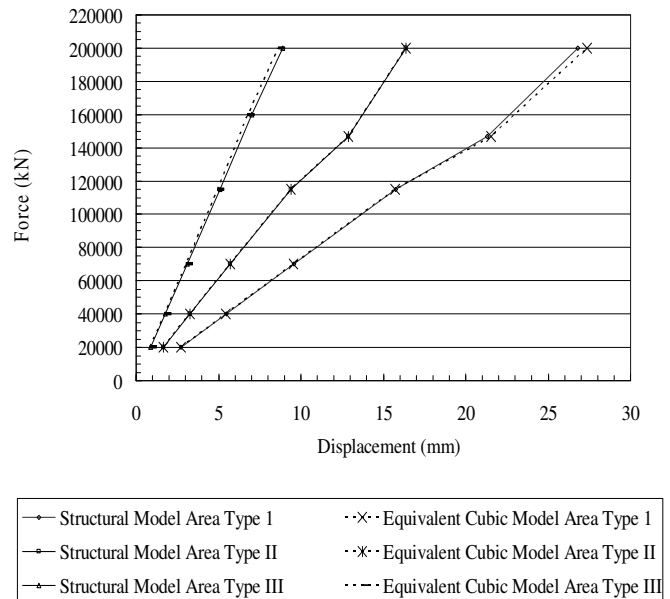


Figure 4: Calibration of F-D Relation of Structural Models and Equivalent Cubic Models

Table 4. Comparison of the Displacements of Structure Models and Cubic Models under Lateral Load

Force (kN)	Displacement (mm)								
	Type I Model	Cubic Model I	Difference I (%)	Type II Model	Cubic Model II	Difference II (%)	Type III Model	Cubic Model III	Difference III (%)
20000	2.730	2.729	0.02	1.332	1.332	0.02	0.886	0.860	2.95
40000	5.465	5.460	0.10	2.665	2.664	0.05	1.774	1.721	2.98
70000	9.572	9.558	0.15	4.666	4.662	0.07	3.105	3.011	3.01
115000	15.747	15.717	0.19	7.669	7.661	0.10	5.104	4.948	3.05
147000	21.296	21.535	1.12	10.509	10.495	0.13	6.995	6.778	3.10
200000	26.800	27.364	2.11	13.351	13.331	0.15	8.887	8.608	3.14

Table 5: Comparison of Displacements of Asymmetric Concrete Core Model and Cubic Model under Lateral Load

Force (kN)	Displacement (mm)					
	Core Model X-Direction	Cubic Model X-Direction	Difference X-Direction (%)	Core Model Z-Direction	Cubic Model Z-Direction	Difference Z-Direction (%)
20000	2.372	2.414	1.78	2.612	2.612	0.00
40000	4.748	4.830	1.72	5.229	5.225	0.07
70000	8.314	8.455	1.69	9.162	9.147	0.16
115000	13.673	13.897	1.64	15.086	15.039	0.31
147000	18.748	19.044	1.58	20.729	20.605	0.60
200000	23.833	24.196	1.53	26.628	26.180	1.68

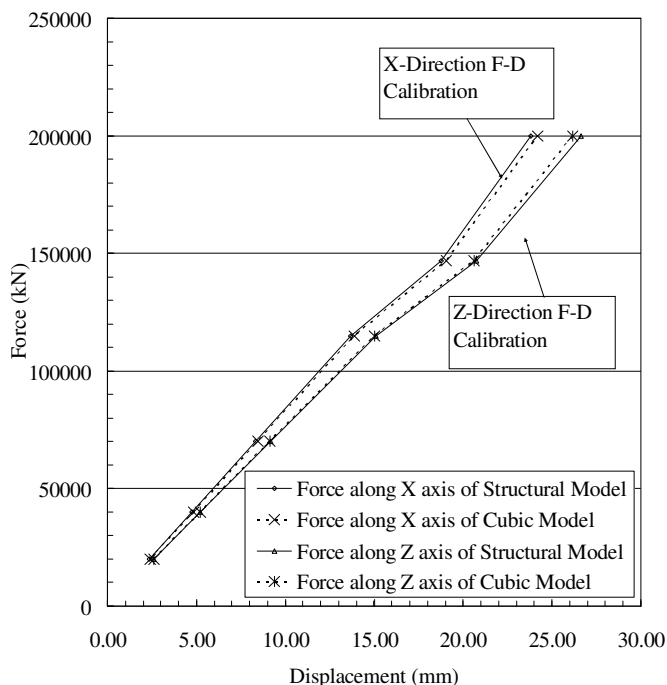


Figure 5: Calibration of F-D Relations of Asymmetric Concrete Core under Lateral Concentrated Load

5 CONCLUSION

This study developed an “Equivalent Cubic Method” to simplify modelling problems when analysing the

static properties of high-rise buildings. A typical 32-storey high-rise building has been modelled with one storey blocks. F-D relationship calibration has been carried out to find the proper simplified cubic model. The following findings have been identified in this study:

- The “equivalent cube method” can be broadly used in static analysis concerned with the serviceability of high-rise buildings. It can efficiently simplify the model and reduce structure dimensions and mesh density and thus reduce the computation time and memory requirements;
- The accuracy of this method appears to be high for the structure analyzed when subjected to a concentrated external force. According to this study, the difference between the real structural model and the equivalent cubic model can be as low as 3%;
- This equivalent cubic method can be extended to the asymmetric structures. Even the asymmetric structure can be simplified using this “equivalent cubic method” and a satisfactory result achieved;
- The equivalent cubic method is beneficial for analysing the influence of non-structural components on the overall performance of high-rise buildings. In using this model, the non-structural components can conveniently be modelled by shell or spring elements connected to the main structural cubes depending on their connection conditions;

- When under pressure or when doing modal testing, owing to the complexity of structural forms and mass distribution, etc. differences between the structural model and equivalent cubic model will appear. So far, according to this study, this equivalent cubic method is not suitable for dynamic analysis.

6 RECOMMENDATIONS FOR FUTURE WORK

Further investigation focusing on the overall behaviour of the structural model built using the equivalent cubic method needs to be conducted to ensure the connection properties between storeys work correctly. Performance of the cubic model with attached non-structural components will also be analysed, as the connection properties and material properties of non-structural components may change with different scaling factors.

REFERENCES

1. Pekau, O.A., Z.A. Zielinski, and L. Lin, Displacement and natural frequencies of tall building structures by finite story method. *Computers & Structures* 1995; 54(1): 1-13
2. Pekau, O.A., L. Lin, and Z.A. Zielinski, Static and dynamic analysis of tall tube-in-tube structures by finite story method. *Engineering Structures* 1996; 18(7): 515-527
3. Oztorun, N.K., E. Citipitioglu, and N. Akkas, Three-dimensional finite element analysis of shear wall buildings. *Computers & Structures* 1998; 68(1-3): 41-55
4. Mahendran, M. and C. Moor, Three-Dimensional Modeling of Steel Portal Frame Buildings. *Journal of Structural Engineering* 1999; 125(8): 870-878
5. Poulsen, P.N. and L. Damkilde, Limit state analysis of reinforced concrete plates subjected to in-plane forces. *International Journal of Solids and Structures* 2000; 37(42): 6011-6029
6. Kim, H.-S., D.-G. Lee, and C.K. Kim, Efficient three-dimensional seismic analysis of a high-rise building structure with shear walls. *Engineering Structures* 2005; 27(6): 963-976
7. Foutch, D.A. and S.-Y. Yun, Modeling of steel moment frames for seismic loads. *Journal of Constructional Steel Research* 2002; 58(5-8): 529-564
8. Hidalgo, P.A., R.M. Jordan, and M.P. Martinez, An analytical model to predict the inelastic seismic behavior of shear-wall, reinforced concrete structures. *Engineering Structures* 2002; 24(1): 85-98
9. Zhang, J. and P.N. Roschke, Active control of a tall structure excited by wind. *Journal of Wind Engineering and Industrial Aerodynamics* 1999; 83(1-3): 209-223
10. Lin, N., et al., Characteristics of wind forces acting on tall buildings. *Journal of Wind Engineering and Industrial Aerodynamics* 2005; 93(3): 217-242
11. Wang, A.-P., Fung, R.-F., and Huang, S.-C, Dynamic Analysis of a Tall Building with a Tuned-mass-damper Device Subjected to Earthquake Excitations. *Journal of Sound and Vibration* 2001; 244(1): 123-136
12. Campbell, S., K.C.S. Kwok, and P.A. Hitchcock, Dynamic characteristics and wind-induced response of two high-rise residential buildings during typhoons. *Journal of Wind Engineering and Industrial Aerodynamics* 2005; 93(6): 461-482
13. Gu, M. and F. Peng, An experimental study of active control of wind-induced vibration of super-tall buildings. *Journal of Wind Engineering and Industrial Aerodynamics* 2002; 90(12-15): 1919-1931
14. Wilkinson, S.M. and R.A. Hiley, A non-linear response history model for the seismic analysis of high-rise framed buildings. *Computers & Structures* 2006; 84(5-6): 318-329
15. Balendra, T., et al., Simplified displacement demand prediction of tall asymmetric buildings subjected to long-distance earthquakes. *Engineering Structures* 2005; 27(3): 335-348
16. Chan, C.-M., and J.K.L. Chui, Wind-induced response and serviceability design optimization of tall buildings. *Engineering Structures* 2006; 28(4): 503-513



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Evaluating the Performance of Low-Cost Inertial Sensors for use in Integrated Positioning Systems

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ABSTRACT

Over the past decade inertial sensor technologies have undergone a significant evolution with regards to their size, weight, power consumption and cost. What is still relatively undefined is the potential of these ‘new’ devices to augment GNSS performance. This task is essential given the growing number of applications that rely on position solutions, combined with an increasing range of positioning accuracy and reliability requirements. This paper presents results obtained from an extensive study undertaken to characterise the performance of current generation inertial sensors. A range of commercially available, low-cost inertial sensors were rigorously evaluated both statically and dynamically. This paper presents a description of the software tool developed to capture the data from all sensors simultaneously and the test platform designed to evaluate the performance of the sensors. A detailed description of the tests performed and the results obtained is also documented in this paper.

KEYWORDS: inertial sensors, MEMS technology, integrated positioning

1. INTRODUCTION

Global Navigation Satellite Systems (GNSS) are currently recognised as the primary technology for the majority of positioning and navigation application. However, while an ever increasing and diverse user community accepts that (under ideal conditions) GNSS can easily achieve the level of performance required, it also widely acknowledges that in certain environments the system can become highly unreliable. Obstructions including tree foliage and buildings impede signals as they travel from the satellites to the receiver, leaving insufficient measurements for positional computations. In addition, where signals do reach the receiver, they may have undergone reflections off surfaces before being received by the GNSS antenna. Such multipathing leads to unknown biases in the satellite-receiver range measurements. And this is simply a fundamental operating constraint of any microwave satellite system (McLellan, 1992).

The all-pervasive influence of GPS has established a trend in international research towards the integration of complementary technologies to remove this constraint, thereby expanding the capabilities of the system. In the majority of cases, the integration philosophy revolves around the augmentation of GPS measurements with dead reckoning (DR) or inertial (INS) systems, using Kalman filtering theory (Cannon et al, 1992). Whilst these systems offer some improvement in the performance of GPS during periods of complete or partial satellite obstruction, in all cases there are practical and theoretical constraints that have hindered their successful implementation.

This paper addresses the practical limitation facing integrated positioning systems in that *the precision of the solution obtained is dependent on the precision of the measurements obtained from the augmentation sensors*. Many engineering applications require continuous position solutions with centimetre level performance. The rapid accumulation of errors in low cost INS and the significantly high cost of more precise inertial sensors have precluded their use in the development of practical integrated solutions for these applications. This research has been inspired by recent progress in surface micromachining technologies, which has facilitated the development of MEMS inertial sensors (DARPA, 2007; Allen et al, 1998). For high precision applications, these developments are significant, as MEMS technologies are now enabling new form factors for inertial sensors. For example, the AGNC-2000 CMIMUTM inertial measurement unit has a volume of less than 16 cubic centimetres and weighs less than 28g. At this stage MEMS technology is still very immature and currently available sensors can only achieve tactical and low-end navigational grade accuracies. However, Sheimy (2000) has indicated that the current trend is towards the rapid development of higher performance MEMS instruments. Combined with cost reductions from tens of thousands of US dollars, e.g. Boeing C-MIGITSTM and Litton LN100TM, to only tens of dollars e.g. Analogue Device ADXL202TM and AGNC-2000 CMIMUTM, the potential of MEMS for improving GPS performance must be investigated. This research takes advantage of the exciting new platform offered by MEMS, to conduct innovative research into integrated positioning systems.

This paper presents a description of the data capture software, the testing strategy and preliminary results obtained from a range of experiments designed to characterise the performance of a range of commercially available MEMS sensors. A performance assessment of these sensors and their use in engineering structural monitoring activities is also presented.

2. METHOD

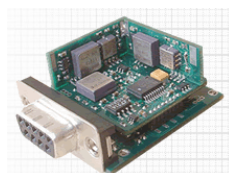
2.1 MEMS Sensors

Table 1 presents a summary of the characteristics of the 4 different MEMS sensors used in this study. These represent the range of commercially available sensors available today. They are Microstrain Inertia-Link™, Crossbow TG™, XSens MTi™, and Cloudcap Crista_IMU (Figure 1). Key design features of these sensors as stated by the manufacturers are listed in Table 1, from which some comparisons can be drawn:

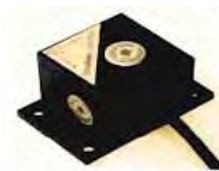
- All the sensors are designed as tri-axial measurement devices.
- With the exception of sensor #2, all others are capable of measuring both tri-axial accelerations and tri-axial gyros.
- According to the specification sheets, all the sensors are designed to be precision instruments with low noise rates.
- Physically, all the sensors are small and light-weight

No	Sensor	Measurement	Range	Error	Sampling Rate	Noise	Size	Weight
1	Crista_IMU	3- Axial Acceleration 3- Axial Gyros	±10g ±300°/sec	<1% <1%	>1KHz		2.05”× 1.55” ×1.00”	36.8g
2	Crossbow TG	3- Axial Acceleration	±2g	±0.0085g	> 200Hz	0.6mg rms	0.98”× 2.235” × 1.435”	110g
3	X-Sens MTi	3- Axial Acceleration 3- Axial Gyros	<2g ±300°/sec	0.02 m/s2 5 °/sec	512Hz 120Hz	0.001 m/s2/ √Hz 0.1 °/sec/√Hz	58mm× 58mm× 22mm	50g
4	Inertia Link	3- Axial Acceleration 3- Axial Gyros	±5g 360°	±0.005g ±0.5° (S) ±2.0° (D)	1~250Hz		41mm× 63mm× 24mm	39g

Table 1: Summary of Key Features of Sensors (From specification sheets)



#1 Crista_IMU



#2 Crossbow_TG



#3 X-Sens MTi



#4 Inertia Link

Figure 1: MEMS sensors

2.2 Design of Sensor Testing

Both static and dynamic tests have been designed and conducted specially for the purpose of evaluating the performance of sensors. This study will focus on three typical standards of sensors performances: repeatability, accuracy, and precision.

To capture the data from the sensors, a data logging software was developed for this research. The Universal Data Logger (UDL) enables all data to be captured simultaneously from all sensors connected to the computer via serial port and USB ports. The sensors are time tagged using either the pulse per second output from a GNSS receiver or with computer time. Figure 2 shows a screen capture from the UDL software. The UDL software also allows the user to specify the output format of the incoming data and to decode the binary data streams if necessary. The measurement data from the sensors are then logged to a Microsoft Access™ database file for subsequent analysis.

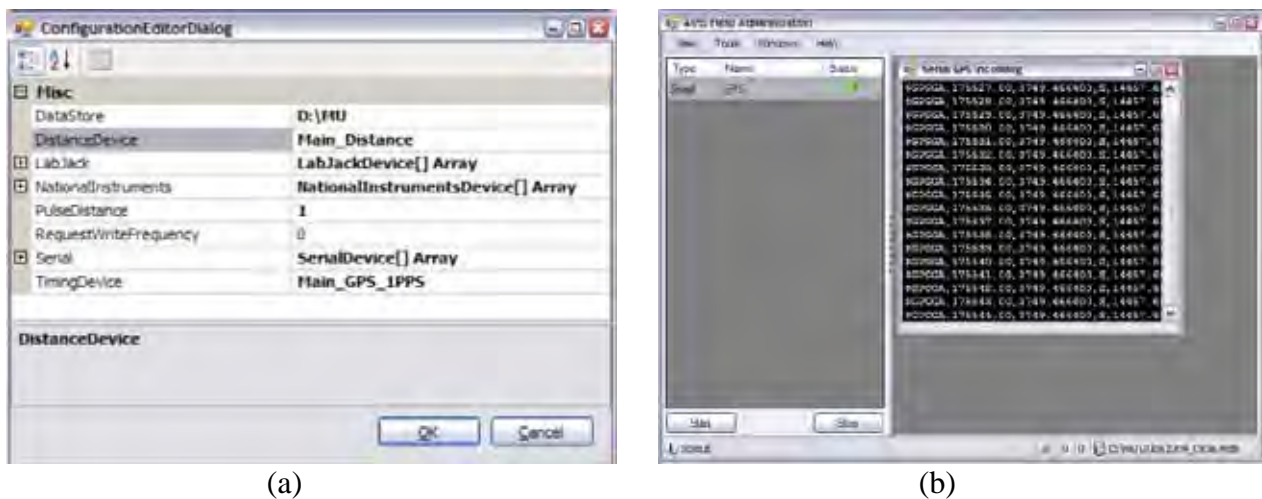


Figure 2: Screen print of the Universal Data Logger software

2.2.1 Static test

Sensors No.1 to No.4 were attached to the test-bed which is a platform fixing to the structural wall of the laboratory. Relatively long measuring period (>24 hours) was designated so that the reliability of the sensors on long-term measurement can be fully validated. Moreover, the test has been repeated three times in order to verify the repeatability of the sensors.

Figure 3 shows the arrangement for static test. The static tests were conducted in a separate lab with restrictions on access of irrelevant people in order to reduce the outside excitation/interruption and simulate static testing environment. However, vibrations of the building itself and some of the interruptions from mid-night cleaning activities were unable to be avoided.

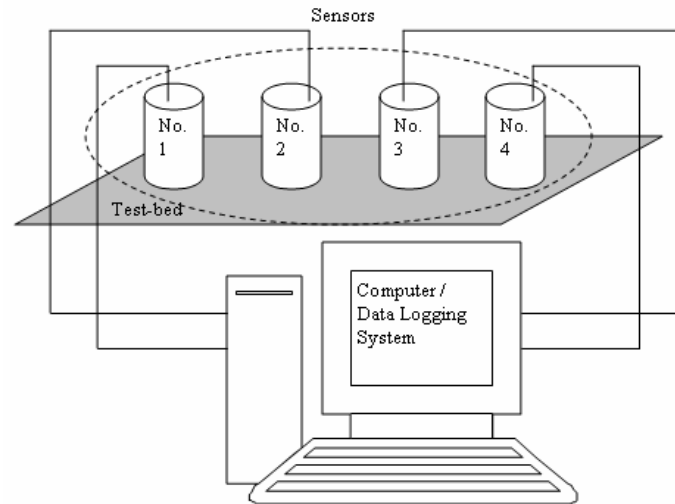


Figure 3: Arrangement of Static Test

2.2.2 Vibration test (Figure 4)

The vibration test was carried out in a structural laboratory by using Tinius machine which can provide constant and controllable vibrations as inputs. Simultaneously, two precise uniaxial accelerometers, Dytran 3191A and 3192A were also used as benchmarks to evaluate the target sensors. Only sensors No.2 and No.4 were tested and No.1 and No.3 were excluded in the test because of the impractical cable lengths and power supply.

During the vibration test, two computing/ data logging systems were involved because of the incompatibility of the two sets of data logging software. Moreover, since the Tinius machine is motivated by hydraulic pressure from the prestored mechanical oil, certain level of instability of the machine performance should be expected.

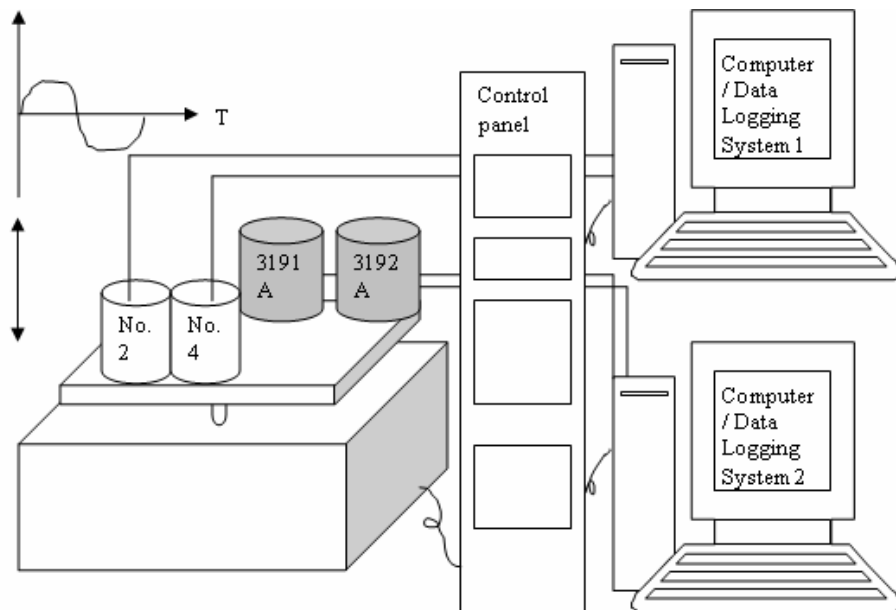


Figure 4: Arrangement of Vibration Test

2.2.3 Assumptions & scope/ limitations of designed tests

Constrained by the testing environment and the resources, there are some limitations and assumptions involved with the tests.

- Restrained by the static testing environment, the tests conducted should be semi-static tests. However, for the purpose of this paper, those results were considered as static testing results.
- Failure of synchronization of the two data logging system induced time lag between sensors. When doing data analysis, the lags were ignored and only key dynamic features of the data (frequency/ period and amplitude, etc.) were analysed and discussed.

3. RESULTS

3.1 Static Test

Possibility density function analysis results for sensors No.1 to 4 are shown in Figure 5 to Figure 16, accompanied by the plotted acceleration-time histories. Error range advertised by each manufacturer was also marked by dashed red lines. Maximum, minimum, and average readings of tri-axial accelerations are extracted from the every-second data and then analysed. To show the measurement clearly, results for three axes were plotted separately.

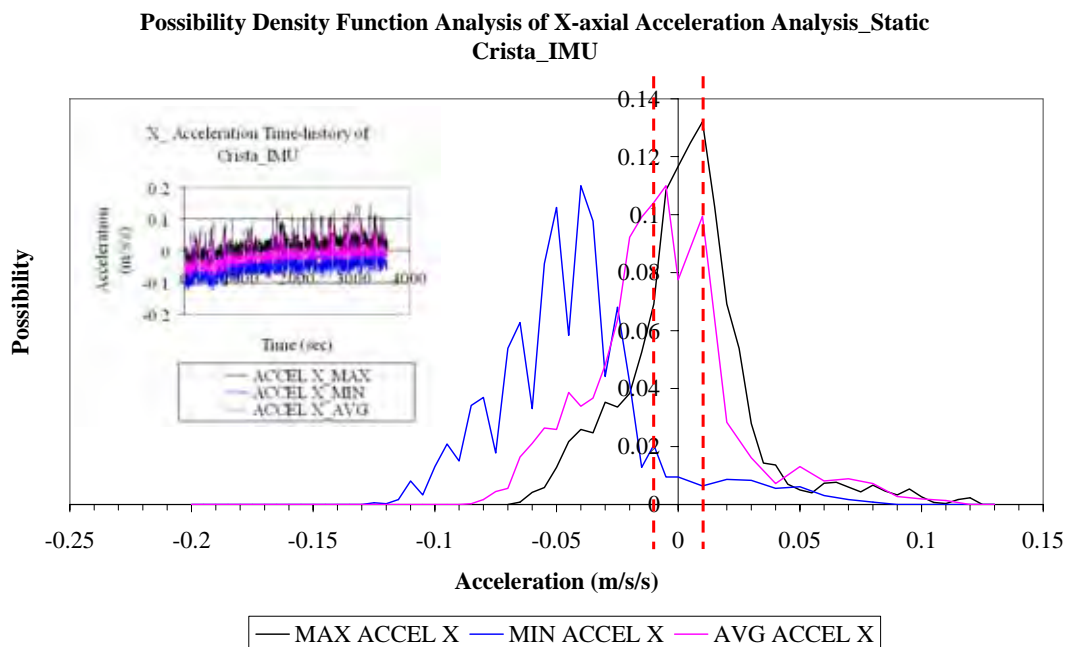


Figure 5: Possibility Density Function Analysis of X-Acceleration_Crista_IMU (Static)

**Possibility Density Function Analysis of Y-axis Acceleration Analysis_Static
Crista_IMU**

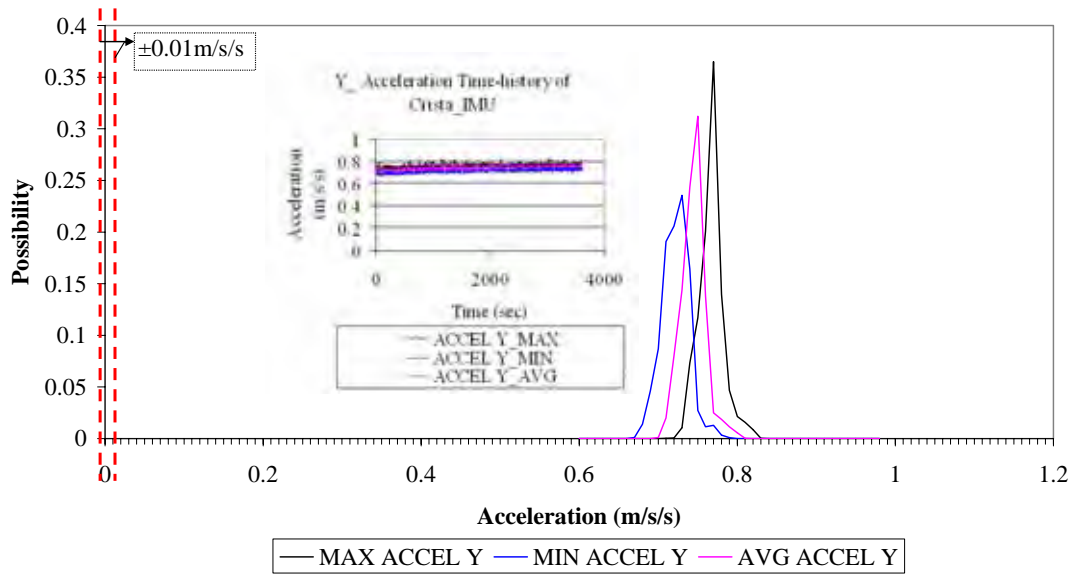


Figure 6: Possibility Density Function Analysis of Y-Acceleration_Crista_IMU (Static)

**Possibility Density Function Analysis of Z-axis Acceleration Analysis_Static
Crista_IMU**

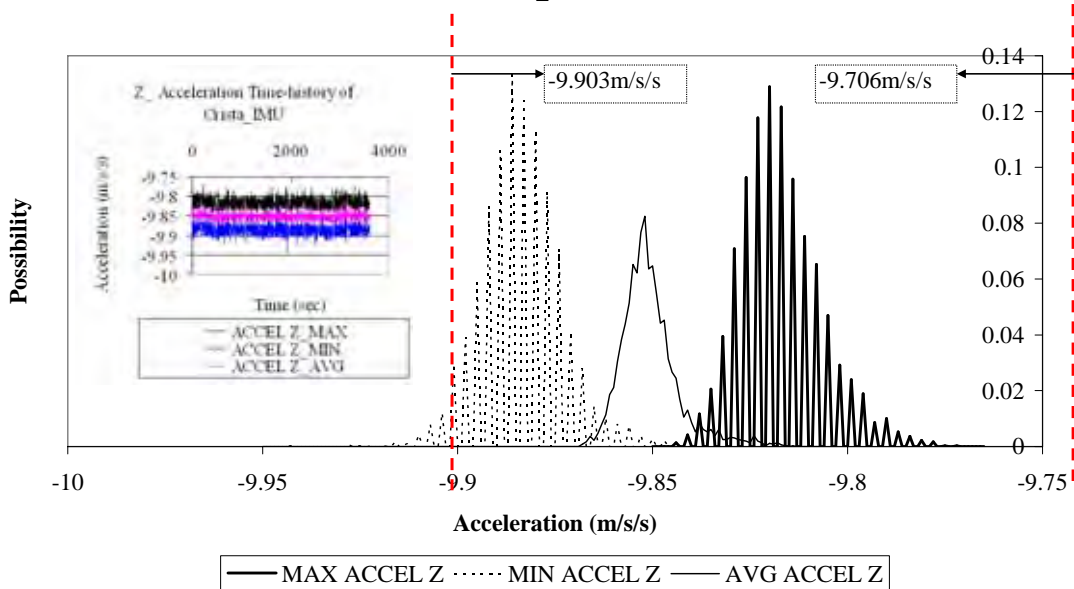


Figure 7: Possibility Density Function Analysis of Z-Acceleration_Crista_IMU (Static)

**Possibility Density Function Analysis of X-axial Acceleration Analysis_Static
Crossbow_TG**

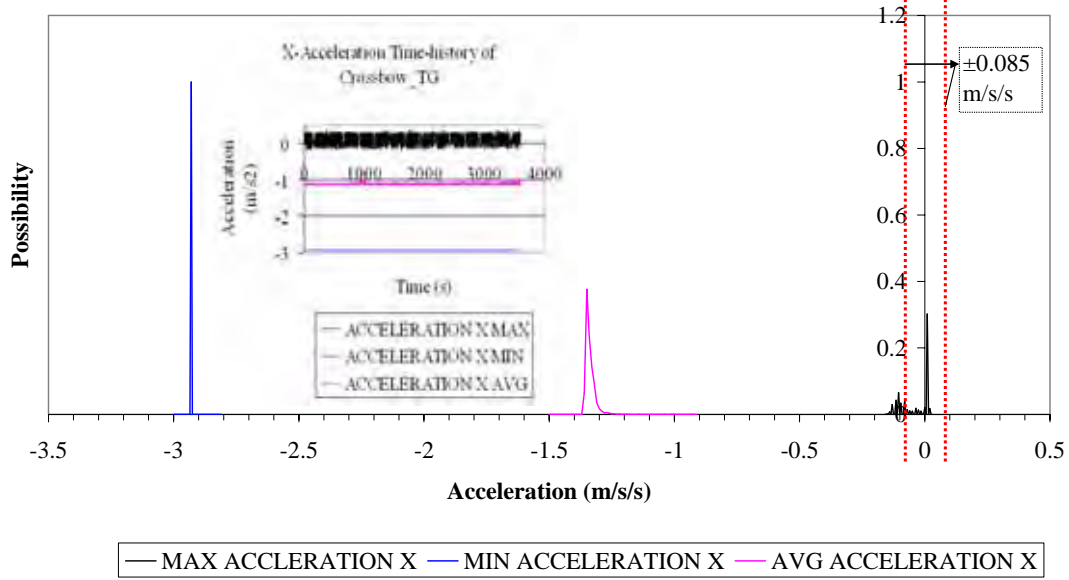


Figure 8: Possibility Density Function Analysis of X-Acceleration_Crossbow_TG (Static)

**Possibility Density Function Analysis of Y-axial Acceleration Analysis_Static
Crossbow_TG**

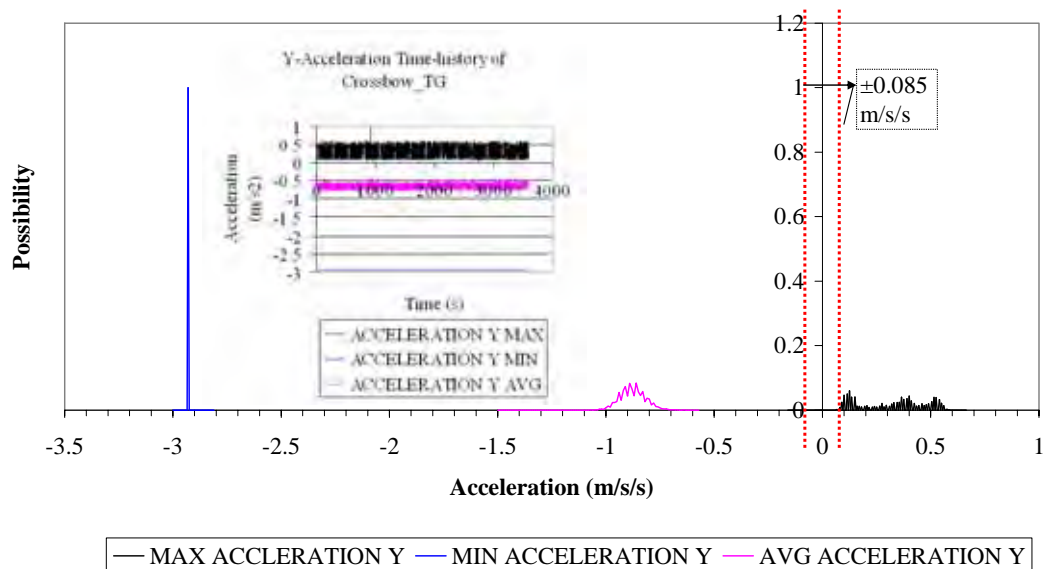


Figure 9: Possibility Density Function Analysis of Y-Acceleration_Crossbow_TG (Static)

**Possibility Density Function Analysis of Z-axis Acceleration Analysis_Static
Crossbow_TG**

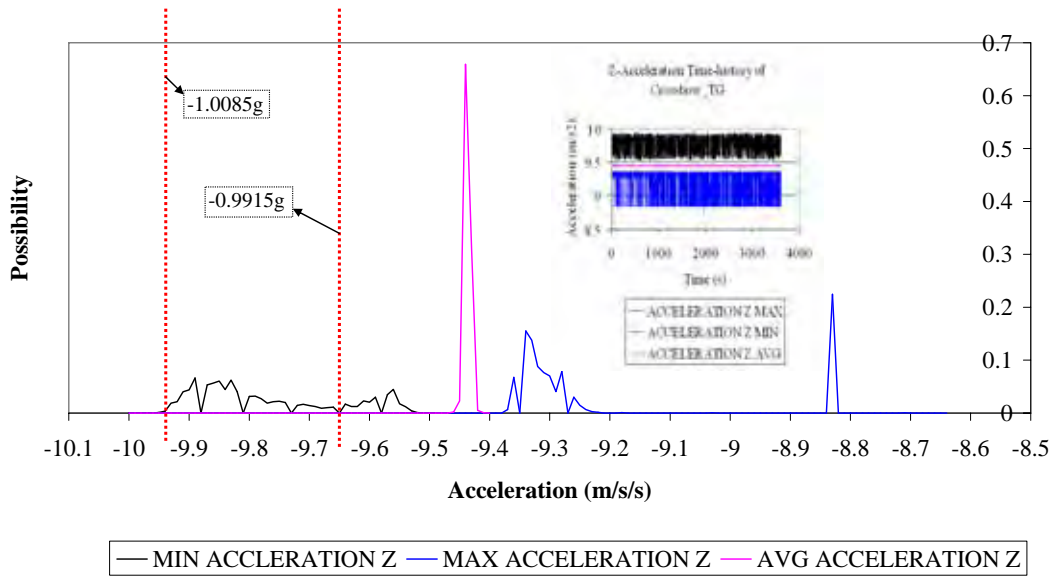


Figure 10: Possibility Density Function Analysis of Z-Acceleration_Crossbow_TG (Static)

**Possibility Density Function Analysis of X-axis Acceleration Analysis_Static
X-Sense MTi**

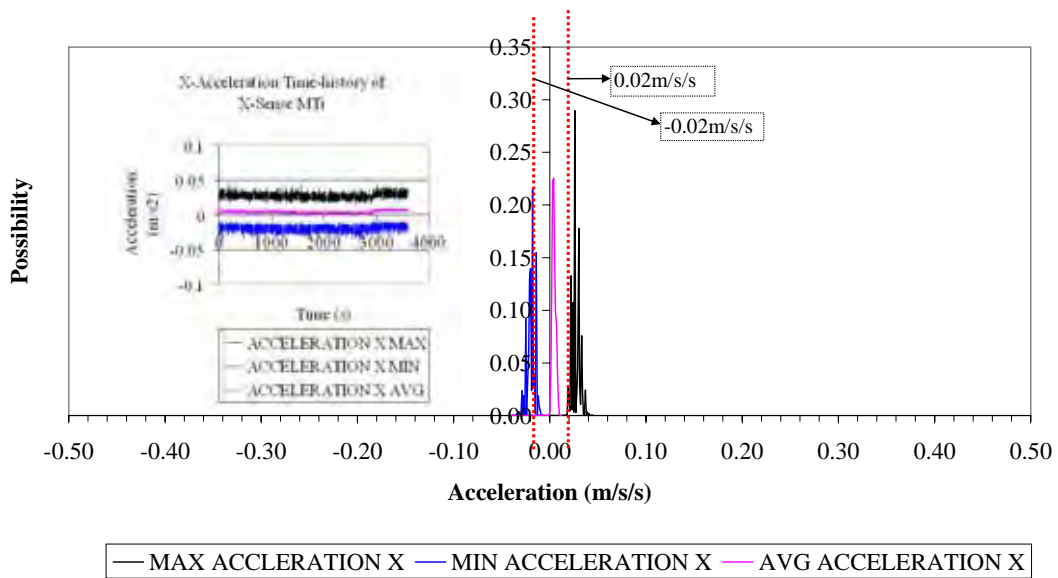


Figure 11: Possibility Density Function Analysis of X-Acceleration_MTi (Static)

**Possibility Density Function Analysis of Y-axis Acceleration Analysis_Static
X-Sense MTi**

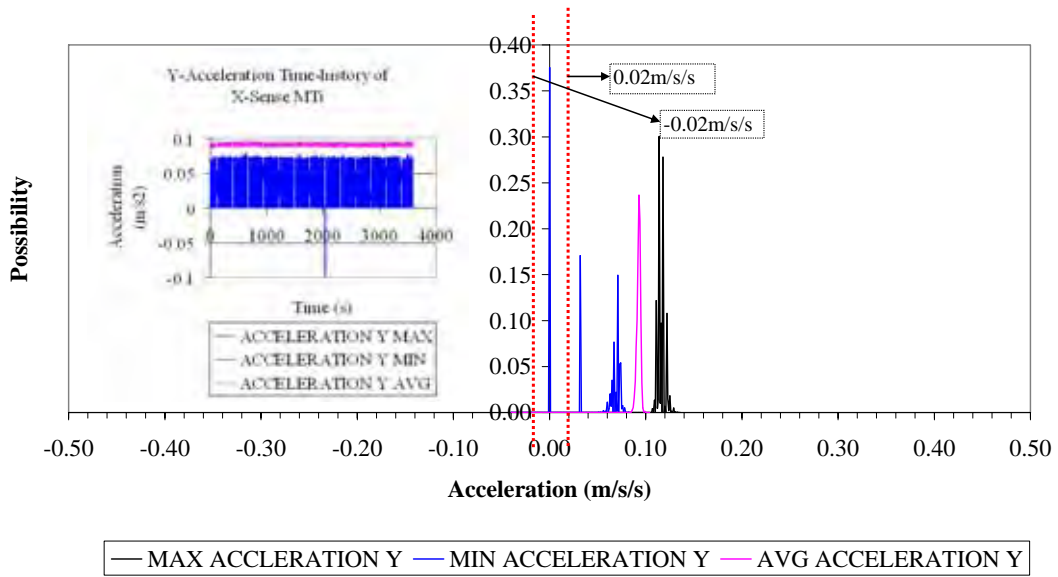


Figure 12: Possibility Density Function Analysis of Y-Acceleration_MTi (Static)

**Possibility Density Function Analysis of Z-axis Acceleration Analysis_Static
X-Sense MTi**

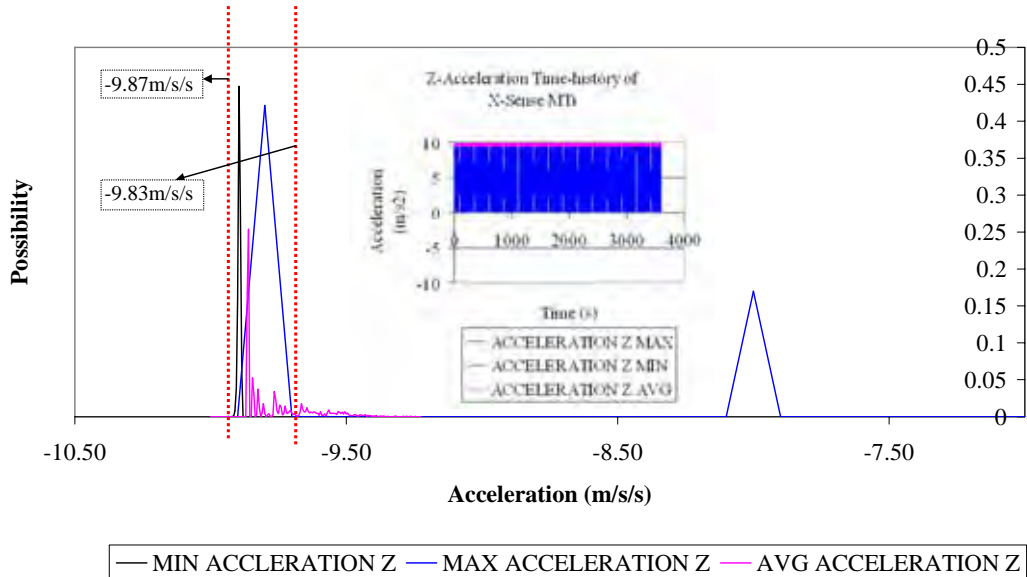


Figure 13: Possibility Density Function Analysis of Z-Acceleration_MTi (Static)

**Possibility Density Function Analysis of X-axial Acceleration Analysis_Static
InertiaLink**

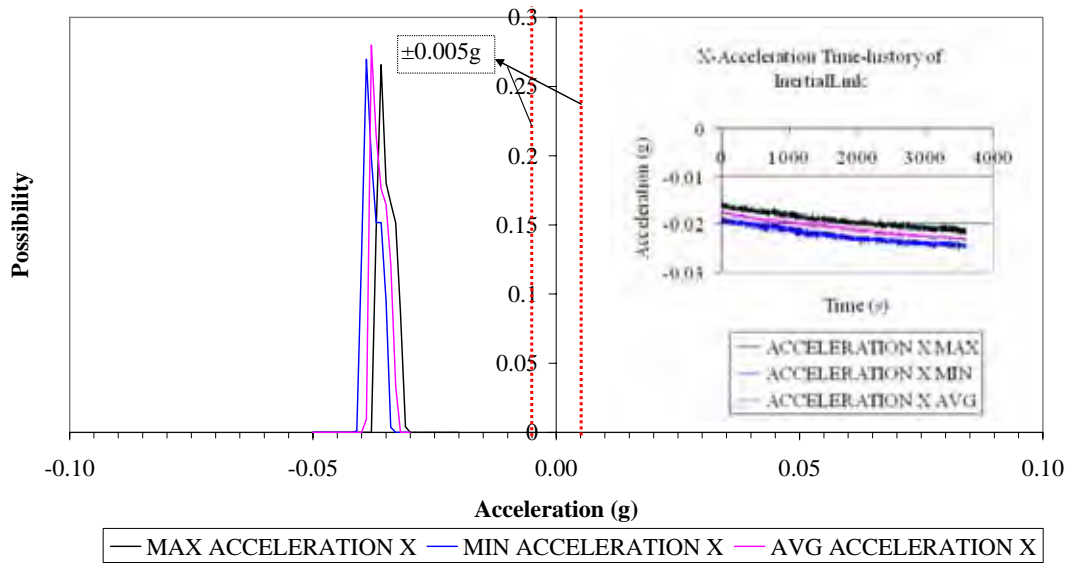


Figure 14: Possibility Density Function Analysis of X-Acceleration_InertiaLink (Static)

**Possibility Density Function Analysis of Y-axial Acceleration Analysis_Static
InertiaLink**

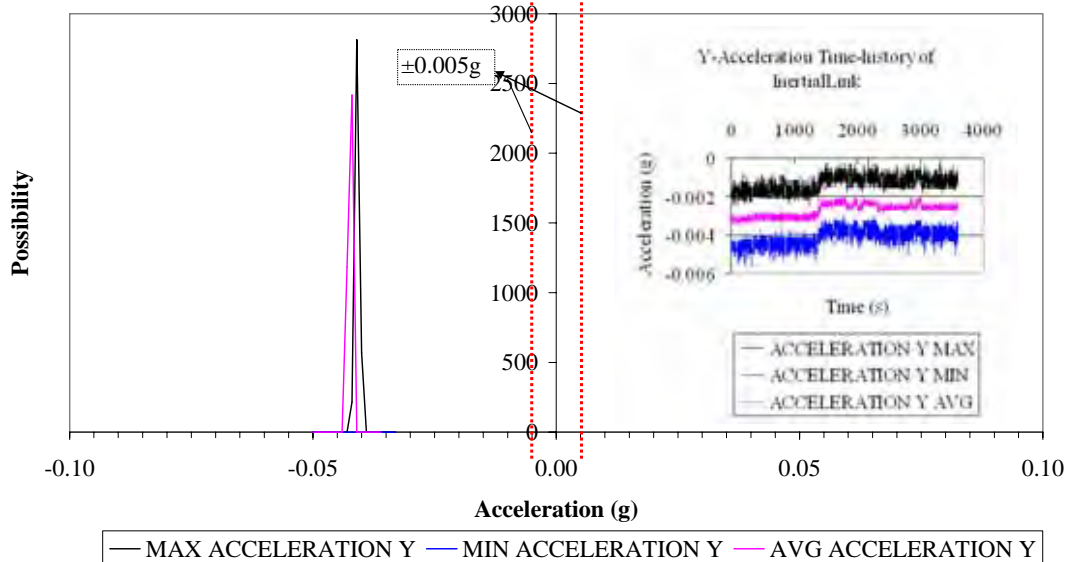


Figure 15: Possibility Density Function Analysis of Y-Acceleration_InertiaLink (Static)

**Possibility Density Function Analysis of Z-axial Acceleration Analysis_Static
InertiaLink**

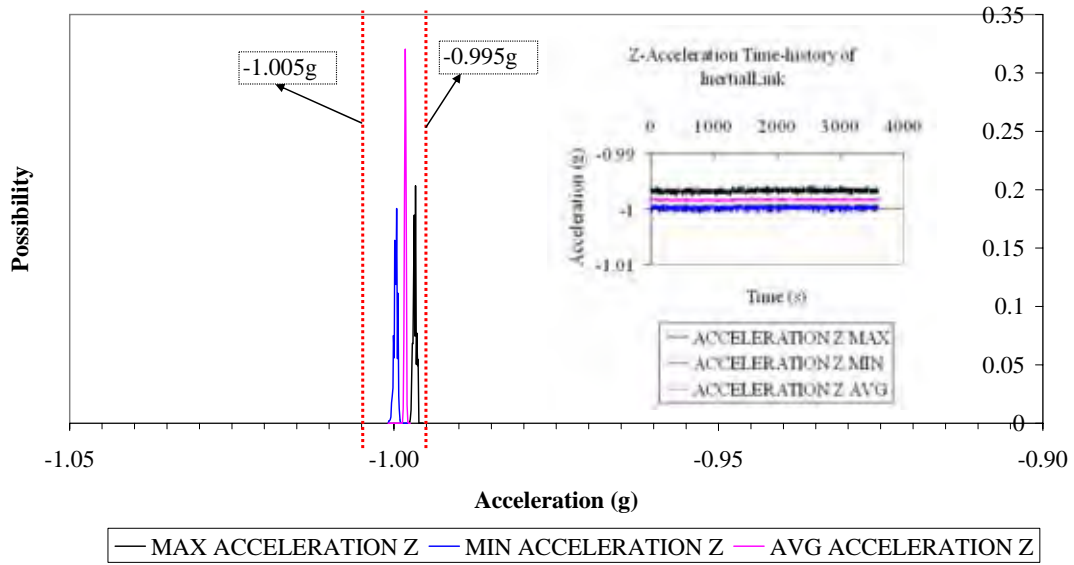


Figure 16: Possibility Density Function Analysis of Z-Acceleration_InertiaLink (Static)

3.2 Vibration Test

Figure 17, 19, 21, and 23 show the comparisons of displacement-time histories analysed from the measurement by the 4 different sensors. They were achieved by doing Fast Fourier Transform (FFT) to the original acceleration-time history records. The selected frequencies are 1Hz, 3Hz, 5Hz, and 10Hz, respectively. Under each frequency, the power spectrum density analysis for every sensor was also conducted so that the reliability of the data captured within different ranges of frequencies can be identified (Figure 18, 20, 22, 24).

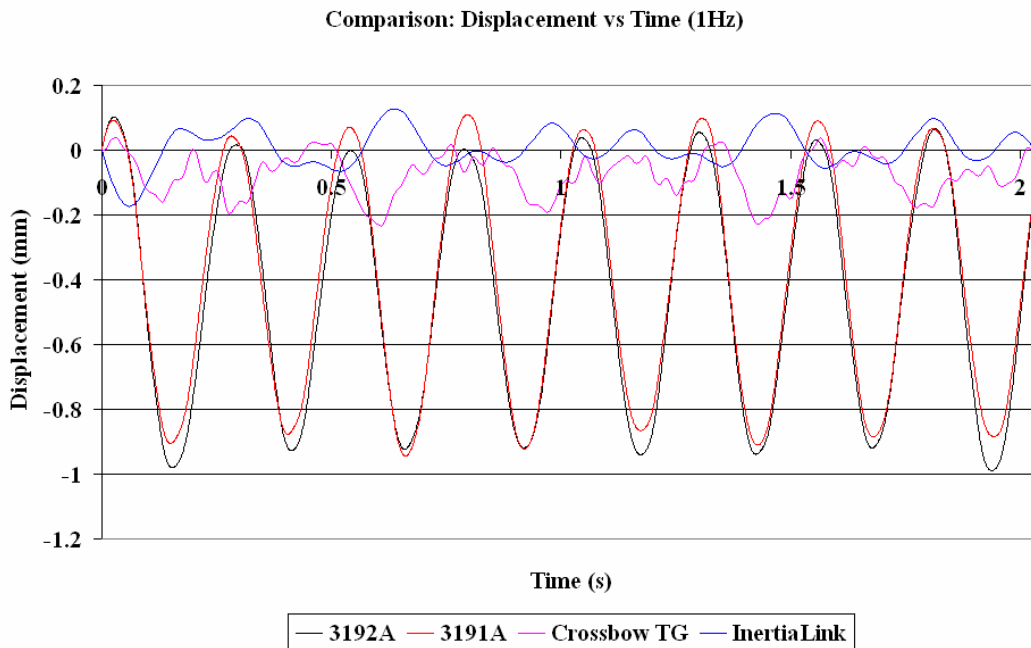


Figure 17: Analysis of Displacement-time Histories at 1Hz

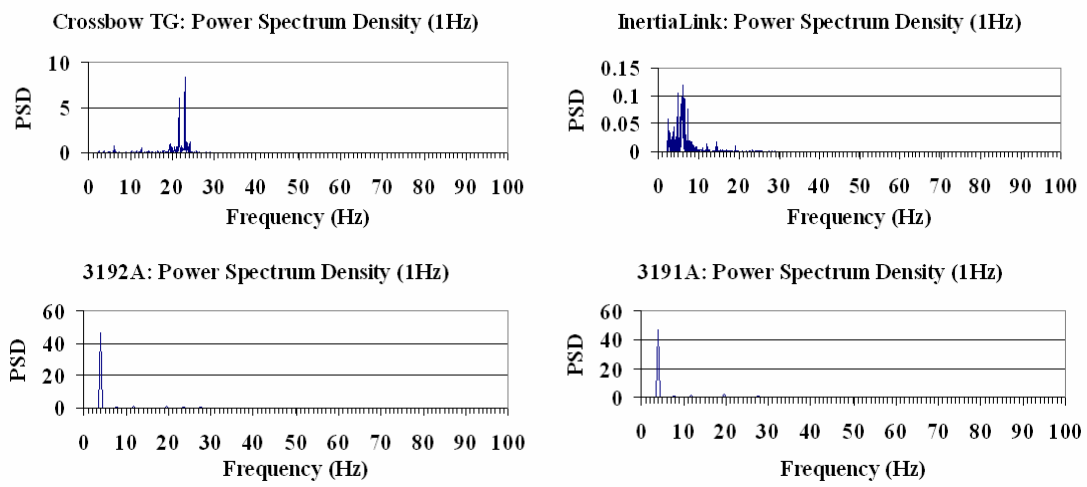


Figure 18: Power Spectrum Density Analysis at 1Hz

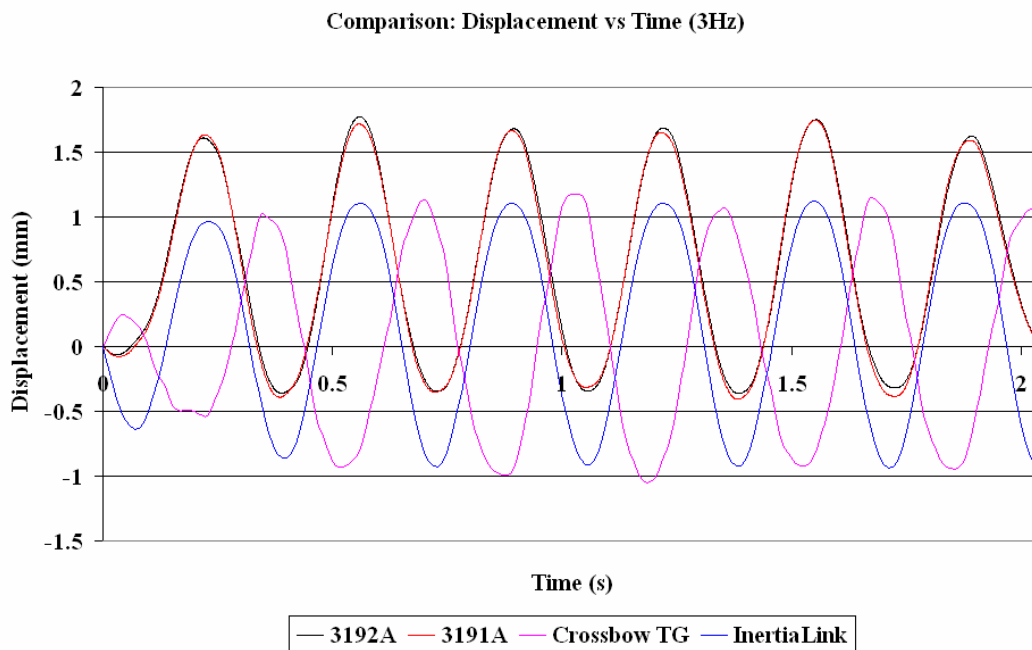


Figure 19: Analysis of Displacement-Time Histories at 3Hz

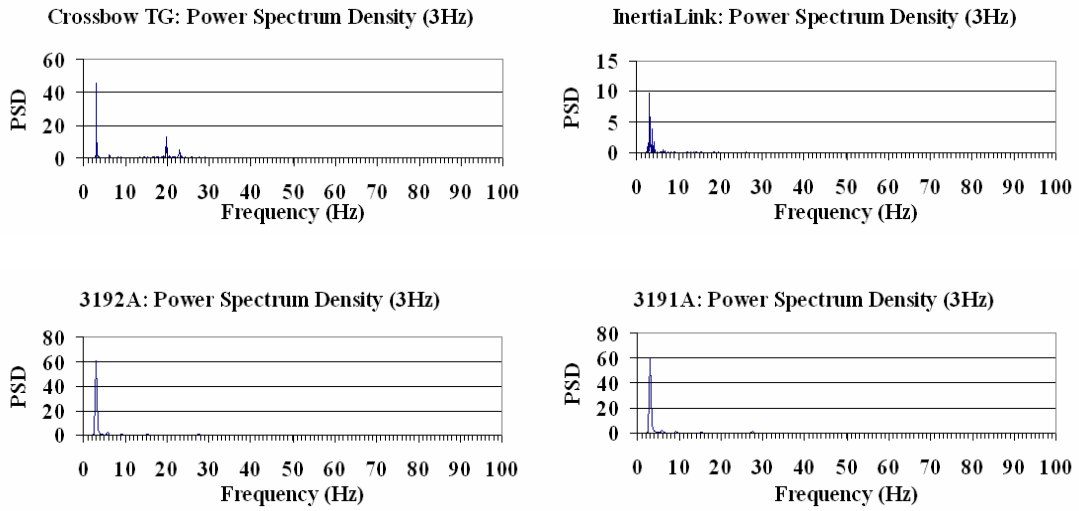


Figure 20: Power Spectrum Density Analysis at 3Hz

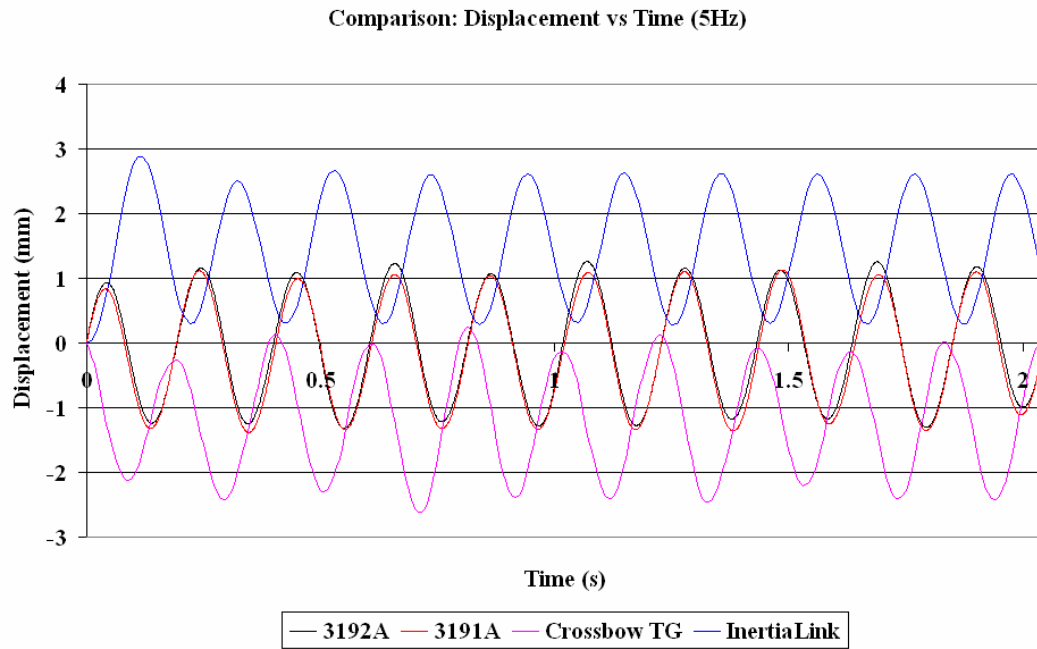


Figure 21: Analysis of Displacement-Time Histories at 5Hz

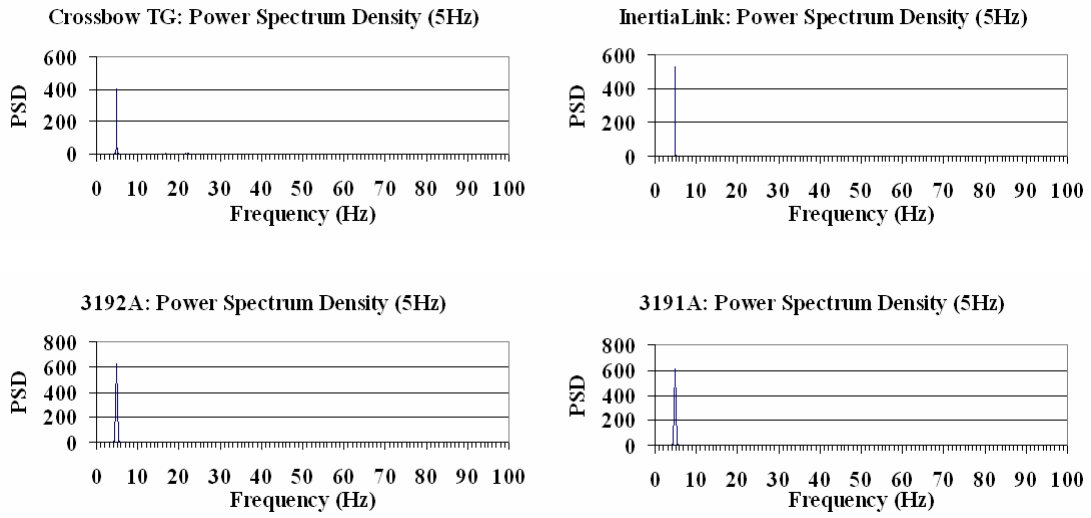


Figure 22: Power Spectrum Density Analysis at 5Hz

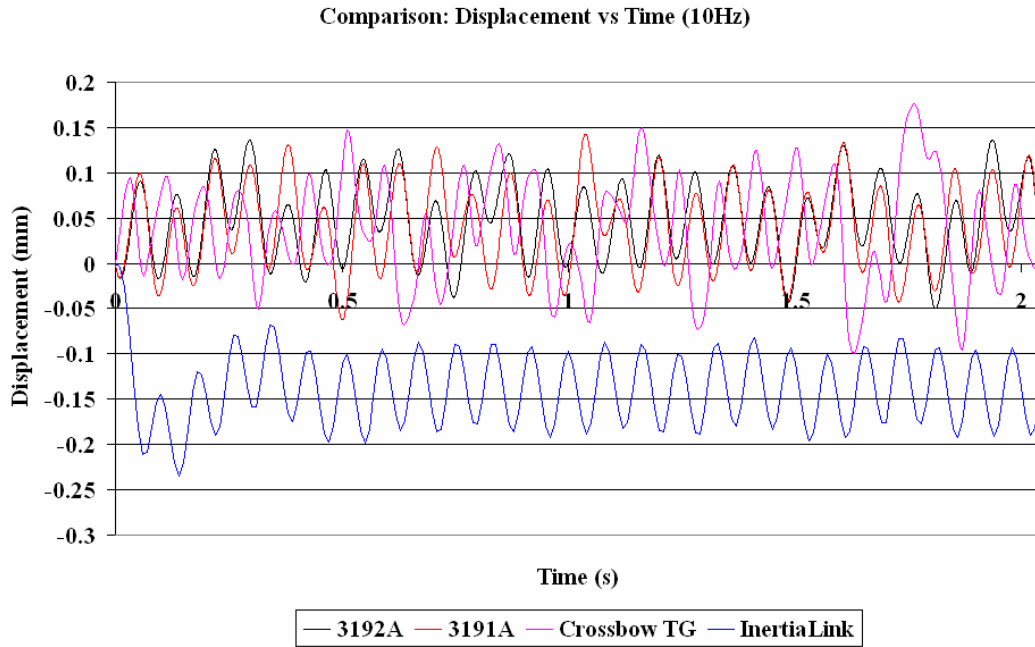


Figure 23: Analysis of Displacement-Time Histories at 10Hz

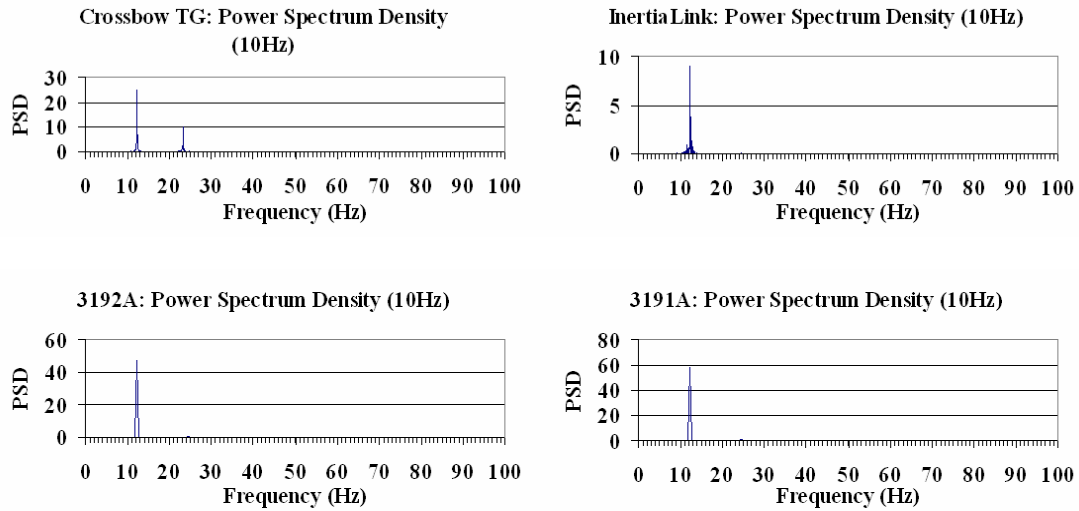


Figure 24: Power Spectrum Density Analysis at 1Hz

4. Discussion

A summary of the performance of the different sensors is presented in Table 2.

No	Sensor	Measurement	Static	Accuracy / Reliability	
				Vibration (<3Hz)	Vibration (≥3Hz)
1	Crista_IMU	X- Axial Acceleration	?	--	--
		Y- Axial Acceleration	N	--	--
		Z-Axial Acceleration	Y	--	--
2	Crossbow TG	X- Axial Acceleration	N	N	Y
		Y- Axial Acceleration	N	N	Y
		Z-Axial Acceleration	N	N	Y
3	X-Sens MTi	X- Axial Acceleration	Y	--	--
		Y- Axial Acceleration	N	--	--
		Z-Axial Acceleration	Y	--	--
4	InertiaLink	X- Axial Acceleration	N	N	Y
		Y- Axial Acceleration	N	N	Y
		Z-Axial Acceleration	Y	N	Y

Notes:

1. "Y" represents yes, which means the sensor can reach its advertised functions
2. "N" represents no, which means the sensor can reach its advertised functions
3. "--" means no comments
4. "?" means no conclusion

Table 2: Summary of performances of different sensors

4.1 Static test

From the results shown in the previous section, the following observations can be obtained for each of the sensors.

Crista_IMU: Figure 5 shows that even though the lower boundary of the x-axial accelerations (down to -0.13 m/s^2) failed to squeeze into the error range ($\pm 1\%$, which is $\pm 0.01 \text{ m/s}^2$) estimated by the manufacturer, some part of the upper limit (up to 0.1 m/s^2) was included in that error range. However, regarding the y-axial acceleration readings (Figure 6), the real measurements (0.68 m/s^2 to 0.84 m/s^2) are far beyond the error limits ($\pm 1\%$, which is $\pm 0.01 \text{ m/s}^2$). Comparing with the x- and y-axial performance, measurement along vertical direction (z-axial) revealed relatively higher accuracy. From Figure 7, it is clear that more than 95% of the readings are allocated within the designed error range (1%, which means from -9.903 m/s^2 to -9.706 m/s^2).

Crossbow_TG: From figure 8, it is clear that only the boundary drawn by the maximum value of x-axial accelerations was mostly allocated within the advertised error range ($\pm 0.085 \text{ m/s}^2$) from the manufacturer. Moreover, the lower boundary drawn by the minimum accelerations of each second (down to -2.9 m/s^2) exceeded the “official” error limitations by more than three times. Regarding y-axial accelerations (Figure 9), both the upper boundary (up to 0.6 m/s^2) and lower boundaries (down to 2.95 m/s^2) are beyond the manufacturer’s specifications. Similarly, along z-axis (Figure 10), only the lower boundary fits in the error range (-1.0085 g to -0.9915 g) while the upper boundary is approximately 0.11 g higher than it.

X-Sense MTi: Figure 11 shows the distribution of the x-axial accelerations of MTi. From the graph, even though both upper and lower boundaries are partly addressed beyond the error range ($\pm 0.02 \text{ m/s}^2$) defined in the specification sheet of the product, it is promising that for the most part the readings are within the range, which means that there is a certain reliability for x-axial measurement of the MTi in the static test. A similar situation occurred along z-axis, which has even better reliability in the readings according to Figure 13. However, for the y-axis, based on the graphs shown in Figure 12, the reliability should be discounted because of the dispersive distribution of the readings (from 0 to 0.12 m/s^2).

InertiaLink: Figure 14, 15, and 16 show the distributions of accelerations along x-, y-, and z-axes respectively. It is clear that only z-axial accelerations are within the error range (-1.005 g to -0.995 g) from the sensor specifications. The x- and y- axes results (-0.04 g to -0.03 g and -0.45 g to -0.39 g , respectively) are not to be sufficiently accurate.

4.2 Vibration Test

Because of the synchronization difficulty, the displacement-time histories have different waves, either sine wave or cosine wave. However, the focus of the vibration tests is on the capabilities of those sensors to capture different motions at different frequencies, as well as the accuracy of the readings. In this case, only the period and the amplitude of the waves are of concern. From Figure 17, 19, 21, 23, it is obvious that the readings from the different sensors started to become consistent when the vibration was equal or higher than 3Hz. It is also worth noting that according to the power spectrum analysis (Figure 18, 20, 22, 24), when the input vibration exceeds 3Hz, both Crossbow_TG and InertiaLink are capable of capturing different level of frequencies with high accuracy.

5. Conclusion

From the above discussion, following conclusions can be drawn about those low-cost sensors:

1. The durability of those sensors is sufficient for most circumstances. Over 24 hours measurements were conducted and the data logging processes all went smoothly.
2. The accuracy of the records from those sensors varies as can be seen from Table 2.
3. The repeatability of the sensors is very satisfactory. The same test was repeated several times and the readings were identical.
4. The precision of those sensors in catching the motion of the object is reliable when the frequency of the object is above 3Hz.

REFERENCES

- Allen J, Sacks E, (1998), Computer-Aided Micro-Mechanism Design, *ASME MEMS Symposium*, USA.
- DARPA, (2007), <http://www.arpa.mil/MTO/Programs/mems/index.html>.
- Cannon M.E, Schleppe J.B, McLellan J.F, Ollevier T, (1992), Real-time Heading Determination Using an Integrated GPS-Dead Reckoning System, *Proceedings of ION GPS-92*, Institute of Navigation, Washington, D.C., pp. 767-773.
- McLellan J.F, (1992), Design and Analysis of a Low Cost GPS-Aided Navigation System, *MSc. Thesis*, 1996, Report No.20097.
- Sheimy N, (2000), Integrated Systems and their Impact on the Future of Positioning, Navigation and Mapping Applications, *Proceedings Quo Vadis International Conference*, FIG Working Week, 2000, 21-26 May, Prague.

The influence of structural and non-structural components on the lateral performance of high-rise buildings

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ABSTRACT: A better understanding of structure performance through the quantification of contributions from secondary elements of high-rise buildings is being investigated as a part of a thrust to optimize the design of high-rise buildings. This paper presents a brief discussion on design development of high-rise buildings and has analyzed current design limitations. Integration of primary and secondary structural components has been identified to be significant to the lateral performance of high-rise buildings. To qualify contributions of different portions, especially non-structural elements such as infill panel and façade to the lateral performance of high-rise structure, quasi-static analysis of models with different structure configurations have been undertaken. In particular, a 32-story building has been studied in detail. The paper concludes with a summary of the contributions made by different parts of high-rise buildings to resist lateral loads and presents influences caused by design and construction incoherence. Limitations of this part of research are also explained, followed by recommendations to future work.

1 INTRODUCTION

Various structural forms and materials are employed in the construction of high-rise buildings. Lateral loads dominant the real performance of high-rise buildings and the structural response depends on not only the structural form and materials but also on the interaction between structural and non-structural components. This paper reports a study into the merit of integrating the contribution from both the structural skeleton and non-structural components such as infill walls and façades to the lateral performance of the building.

1.1 *Design development of high-rise buildings*

High-rise building design has evolved considerably since the early buildings of the mid 1900s to current structures such as the Taipei 101 building. The stages of evolution have always sought to make structures stronger, safer, more efficient, and more comfortable. Typically, development of high-rise buildings can be illustrated using the development of high-rise structure forms as depicted in Table 1 and Figure 1.

The building height has been constrained by technical aspects such as the structural form and construction materials. When the structure is tall enough, the influence from lateral loads such as wind and earthquake frequently dictate the design outcomes

when complying with either ultimate limit state or serviceability requirements.

1.2 *Current design analysis and limitation*

Since World War II, the design philosophy and international standards have shifted their emphasis from working stress design approaches to limit state design based on generally accepted probability-based approaches. The limit state approach aims to ensure the structure and its constituent components are designed to resist the worst loads and deformations during the construction and service, with reasonable safety and to have adequate durability during their life (Stafford & Coull 1991). The ultimate limit state and serviceability limit state are now used in governing the performance of the entire structure, or any part of it.

The design process of typical 30 to 50 story buildings involves designing a skeleton to resist the ultimate limit state loads and the serviceability limit state loads, including allowance for winds and earthquakes. It is a common practice that engineers ignore the structural effect of non-structural elements such as partitions; doors; windows; façades; etc. by considering those non-structural components solely as they are isolated from the skeleton.

However, buildings are widely recognized as a complex assemblage of both structural skeleton and non-structural components (Su, et al., 2005).

Table 1. Development of structure forms and lateral resisting system.

Form	Lateral force resisting members	Advantages	Preferred material	Height tolerance
Braced-Frame Structure	Diagonal members girders, vertical truss, columns	<ul style="list-style-type: none"> Highly efficient in resisting lateral loads High stiffness for a minimum of additional material 	Steel	Very tall
Rigid-Frame Structure	Columns, girders, and connections	<ul style="list-style-type: none"> Open rectangular arrangement allows freedom of planning and easy fitting of doors and windows 	Reinforced concrete/ Steel	Up to 25 stories
Infilled-Frame Structure	Infill	<ul style="list-style-type: none"> Hard to predict the stiffness Complex interactive behavior 	Reinforced concrete/ Steel	Up to 30 stories
Flat-Plate and Flat-Slab Structure	Similar to rigid-frame structure, and flat-plate or flat-slab	<ul style="list-style-type: none"> Simplest and most logical structure form 	Reinforced concrete	Up to 25 stories
Shear Wall Structure	Continuous vertical walls	<ul style="list-style-type: none"> Very high in-plane stiffness and strength 	Reinforced concrete/ Masonry Concrete	Up to 35 stories
Wall-Frame Structure	Shear wall and rigid frame	<ul style="list-style-type: none"> Stiffer and stronger 		40 to 60 stories
Framed-Tube Structure	Tube formed by very stiff moment resisting frames	<ul style="list-style-type: none"> High structural efficiency Appropriate for use up to the greatest of heights 	Steel/ Reinforce concrete	40 to more than 100 stories
Outrigger-Braced Structure	Outrigger trusses, braced core, columns	<ul style="list-style-type: none"> Efficient Provides greater construction height 	Steel/ Reinforce concrete	40 to 70 stories
Suspended Structure	Cantilever trusses, braced free of core, hangers	<ul style="list-style-type: none"> The ground floor can be entirely major vertical members Some construction advantages 	Steel/ Reinforce concrete	Not specified
Core Structure	Single core	<ul style="list-style-type: none"> Similar to the suspended structure Inefficient in resisting lateral loading and supporting the floor loading 	Steel/ Reinforce concrete	Not specified
Space Structure	A three-dimensional triangulated frame	<ul style="list-style-type: none"> High efficient Relatively light weight Potential to achieve the greatest height 	Steel/ Reinforce concrete	Not specified
Hybrid Structure	Combination of two or more above structure forms			

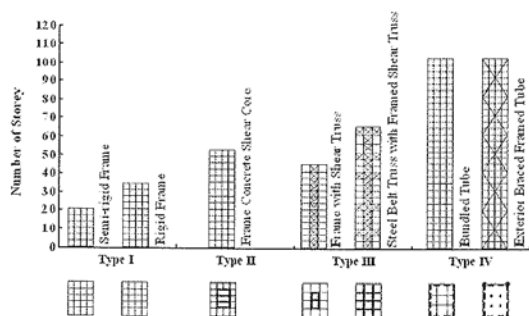


Figure 1. Development of the height and structure form of high-rise buildings.

The complexity of lateral performance of high-rise building is even remarkable because of the conflicting requirements of diverse (structural and non-structural) building system (Hutchinson, et al., 2006). There is scope to improve the serviceability limit state design requirements over the traditional approach.

The so-called non-structural components and the interaction between structural and non-structural components play important roles in influencing lateral performance of high-rise buildings (Mahendran & Moor 1999, Sev 2001). Due to insufficient consideration to the interaction between different structural systems the design requirements for internal structural members may become unreasonable (Kuang & Li 2005). According to previous research, the structural role of non-structural components such as façade, infill walls,



Figure 2. Case study building under construction.

etc. in resisting the lateral loads is significant and can add on the overall building stiffness up to 87% (Melchers 1990, Arnold 1991, Hall 1995, Gad et al., 1998, 1999a, 1999b, Naeim 1999, Hoenderkamp & Snijder 2003, Su et al., 2005).

1.3 Overview of this study

To better understand the lateral performance of high-rise buildings, it is necessary to find out the contribution from different structural systems. This study sought to identify the importance of typical non-structural components of high-rise buildings in resisting lateral loads. Finite element (FE) models have been developed for quasi-static analysis of a 32-storey building and ascertain the Force-Displacement (F-D) relationships of the:

- structural frame;
- structure frame with infill walls;
- structure frame with façade panels; and
- structure frame with infill walls and façade.

2 FIELD OBSERVATIONS

A case study building in Melbourne, Australia, was adopted for detailed investigation. This building is designed and constructed as a typical reinforced concrete structure with structural and non-structural facades for residential purpose (Fig. 2). Specific details of the case study project follow.

2.1 Material

Major materials used in this project are reinforced concrete, precast concrete and steel. Masonry, timber and glass have been used for non-structural components.

Table 2. Element details of the case study building.

Element	Cross section (mm ²)	Thickness (mm)	Re-bar Diameter (Mm)	Concrete grade (MPa)
Beam	T- L-		N16~N36	32
Column	Rectangular		N16~N36	80~32
Floor slab		190	N12~N36	32~40
Shear wall		150~400	N16~N36	

2.2 Building details

2.2.1 Structural elements

The structural elements in this building include a core, shear walls, spandrel beams and columns. All the element details follow the requirements of AS/NZS 3600. Details of those elements are listed in Table 2.

2.2.2 Non-structural elements

The non-structural elements referred to are designed as non-load bearing elements and they include infill walls, façade panels, doors, windows, etc. All the details of the non-structural components are in accordance with AS/ NZS 3600, 3850, 1554.

Infill walls in the case study building are precast concrete walls and brick infill walls, such as stairwells.

Double-skin modern façade technique has been employed by using glass with different colours and aluminium frames.

Doors and frames in this project are made of either timber or metal. Windows are formed by single-or double-skin glass together with metal frames. Owing to its residential purpose, most of doors and windows in this building have standard dimensions and properties. Full details of the constructed form of this building are presented in Li et al. (2007).

2.2.3 Connections between elements

Generally, connections in the building can be divided into three categories according to different types of elements:

- Connections between structural elements. This means the connection among beams, columns, walls and floor slabs;
- Connections between structural and non-structural elements. This type of connections focuses on the interface between structural elements and non-structural elements, that is, how the non-structural elements have been connected to the primary structure.
- Connection within the non-structural elements. These are the details of the connection between different parts of non-structural components.

3 MODELLING APPROACH

Detailed FE models have been developed to analyse the structure performance. In accordance to the general approach (Mahendran & Moor 1999, Grierson & Khajehpour 2002, Kicingier et al., 2005, Sarma & Adeli 2005, Wu et al., 2006), 3-D models are accepted as more accurate than 2-D models (Mahendran & Moor 1999, Grierson & Khajehpour 2002, Kicingier et al., 2005, Sarma & Adeli 2005, Wu et al., 2006), hence, a 3-D approach was adopted.

3.1 Software

The commercial FE analysis package ANSYS 10.0 (University Introduction Version) has been employed as the analytical tool to simulating and calculating the F-D relations of different structure configuration under quasi-static loading conditions. There are two main advantages in choosing this software package:

- Various of elements can be used to define different structure systems; and
- Complex enough for finite element analysis of non-linear large displacement performance.

3.2 Assumptions

3-D FE analysis of a complex high-rise building carried out by ANSYS is computationally demanding. The following modelling assumptions were made:

- symmetry in the building has been used to scale down the model;
- the building frame has been simplified to regular shape;
- frame elements have been approximated to a common cross-sections;
- a unified grade of concrete has been adopted;
- there is no special consideration for connections.

The assumptions are considered reasonable as the purpose of this study is to identify the influence brought by non-structural components, thus, the error from frame assumptions will not greatly influence the findings of this comparative analysis. In respect to connections, there should be another study specifically focusing on the details.

3.3 Material properties

Concrete and steel are the two types of material used in the models. Concrete was used for structure frame and infill walls (Fig. 3), while steel was for connections and non-structural façades (Table 3).

3.4 Element properties

Beam element, shell element, and spring element are used to simulate the structural frame, façade, infill walls, and connections respectively. The cross section

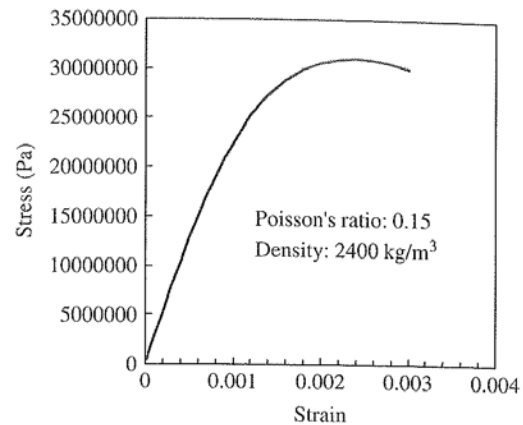


Figure 3. Concrete properties used in modelling.

Table 3. Steel properties used in modelling.

	Yield stress MPa	Young's modulus MPa	Poisson's ratio	Tangent modulus	Density kg/m ³
Steel	640	2×10^{11}	0.299	2×10^{10}	7850

of beam element is a $0.3 \text{ m} \times 0.3 \text{ m}$ square. The shell element for infill walls is 0.15 m thick while the shell element for façade panels is 0.002 m . A stiffness value k of 50 N/m has been given to the connections between frame and façade panels, since it is explained in the assumption that the influence caused by the stiffness between connections won't be included in the consideration of this paper.

3.5 Boundary and loading conditions

In all models, 6 degrees of freedom of the frame base have been constrained, i.e. the frame of the structure is just like a cantilever. Displacements and rotations along x , y , z directions at the bottom of infill walls were coupled to the same storey of frame. To achieve the quasi-static loading condition, lateral loads along x direction at the top of building were applied gradually in small load steps.

3.6 Modelling scenarios

To clearly identify the different contributions from infill walls and façade panels to the lateral performance of high-rise building; the following scenarios have been designed for the quasi-static analysis under lateral loads (Fig. 5):

- frame only (Model a)
- frame + infill walls (Model b)

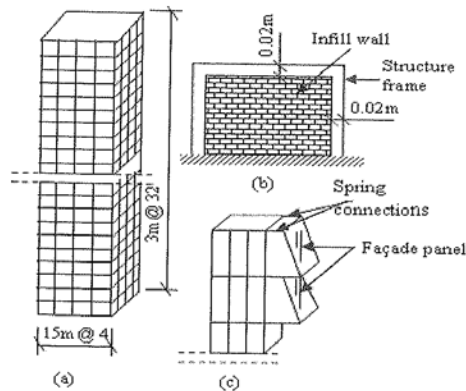


Figure 4. Model details.

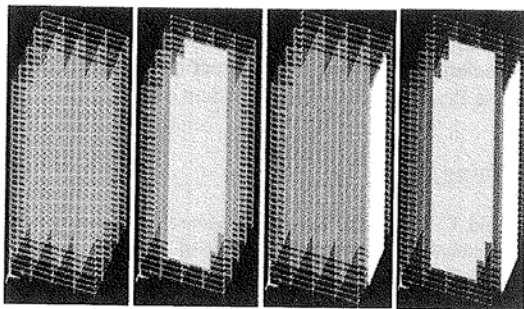


Figure 5. Model details (a) frame (b) frame + infill (c) frame + façade (d) frame + infill + façade.

- frame + infill walls along lateral load direction
- frame + infill walls across lateral load direction
- frame + façade (Model c)
 - frame + façades along the lateral load direction
 - frame + façades across the lateral load direction
- frame + infill walls + façade (Model d)
 - frame + infill walls + façades along the lateral load direction
 - frame + infill walls + façades across the lateral load direction.

4 RESULTS

The results from the above four scenarios have been plotted in Figures 6–8. The F-D relation from model b, c, and d are compared with the result from Model a (Figs. 6, 7, 8) to find the difference between these contributions in resisting lateral loads.

5 DISCUSSION

From the results, it is clear that infill walls, even if they are not structurally connected to the frame, will

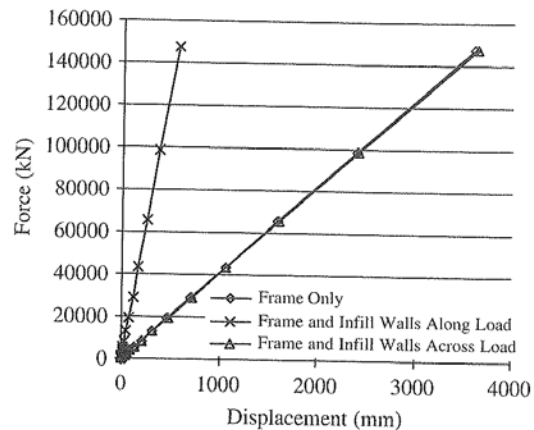


Figure 6. Comparison of deformation capacity with and without infill walls.

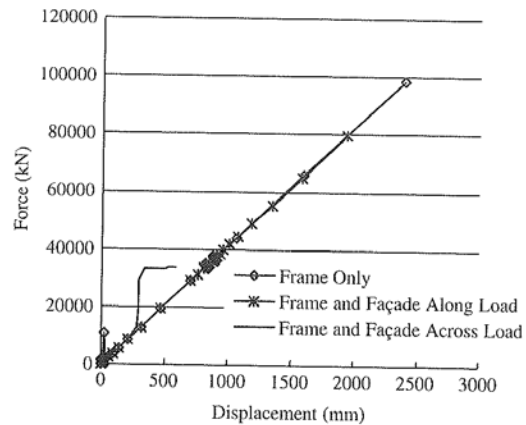


Figure 7. Comparison of deformation capacity with and without façade panels.

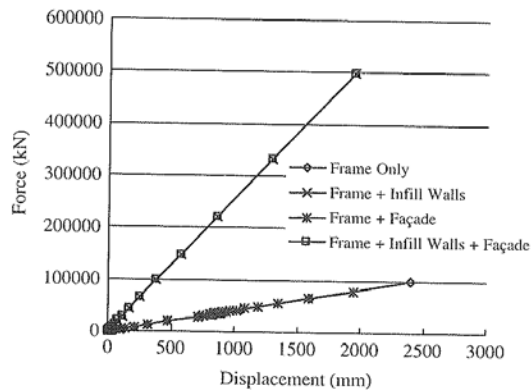


Figure 8. Comparison of deformation capacity with different non-structural components.

have a significant contribution in resisting lateral loads which can increase up to more than 85% of the total building stiffness. It is also interesting to note, there is little influence on building performance when the infill walls are located across lateral load direction. It is also concluded that the stiffness contribution from infill walls alters with changing locations.

The stiffness contribution from façade panels is not as obvious as that of infill walls. From the figures, the deformation capacity of the building kept unchanged when installed façade panels both along and across lateral load directions. This indicated that under normal connection condition, façade panels won't have significant influence on the lateral load resisting capacity of buildings, wherever their locations are. From Figure 7, it can be observed that the load bearing capacity of the building frame decreased when the facade panels are connected to the main building by steel springs with k value of 50. There are two reasons which can be expected to explain this phenomenon, namely:

- The interaction between facade panels or between panels and frame may provide a negative influence to the lateral behaviour of the building. The elements, especially connecting elements, of the building may be severely damaged much earlier than the whole building reaching to its upper load bearing limitation. For this situation, at the time the element is damaged, the software package will stop calculation and give the solution that the displacement of the building has exceeded the limitation.
- The complexity of connection properties may cause a decrease of a buildings load bearing capacity. Further detailed static analysis of the building lateral performance with façade panels under different connection properties is required to confirm the extent of this decrease.

When both façade and infill walls are installed, the comparison of F-D relationships shows in Figure 8 confirms the expected behaviour. The combined effort is exactly the sum of those two separate ones.

6 CONCLUSION

The following conclusions have been drawn from this study.

- Infill walls have a significant contribution to the lateral deformation capacity of high rise structures when they are located in the along-load direction. At one extreme structure stiffness could be improved by up to 85%;
- The location of infill walls has a critical influence their contribution to the building lateral performance. When the infill walls are located in the across-load direction, there won't be distinguished improvement of building stiffness;

- Façade panels attached to the external wall of buildings do not greatly influence the structure stiffness, regardless of their location;
- façade panels may reduce the load bearing capacity of the whole building depending on the connection properties;
- the combined effect of infill walls and façade panels is exactly equal to the sum of both parts.

In summary, the contributions from non-structural components to the lateral performance of high-rise buildings are notable enough to be analysed and considered carefully in design and construction process.

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REFERENCES

- Arnold, C. 1999. The Seismic Response of Non-structural Elements in Buildings, *Bulletin of the New Zealand National Society for Earthquake Engineering*, 24(4), pp 306–316.
- Gad, E.F, Duffield, C.F., Chandler, A.M. & Stark, G., 1998. Testing of Cold-formed Steel Framed Domestic Structures, the 11th European Conference on Earthquake Engineering, France, CD Rom.
- Gad, E.F, Chandler, A.M., Duffield, C.F. & Stark, G., 1999. Lateral Behavior of Plasterboard-Clad Residential Steel Frames, *Journal of Structural Engineering* 125(1): 32–39.
- Gad, E.F, Duffield, C.F., Hutchinson, G.L., Mansell, D.S. & Stark, G., 1999. Lateral performance of cold-formed steel-framed domestic structures *Engineering Structures* 21(1): 83–95.
- Hutchinson, G.L., Collier, P., Duffield, C.F., Gad, E.F, Li, B. & Mendis, P. 2006. Integration of GPS measurements and secondary structural elements into the serviceability design of structures, The Tenth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-10), Kanak-Nukulchai, W, Munasinghe, S and Anwar, N (Eds), Bangkok, Thailand, 3–5 August, ISBN 974-8257-25-8, Asian Institute of Technology, Supplementary Papers, pp13–20.
- Grierson, D.E. & Khajehpour, S., 2002. Method for Conceptual Design Applied to Office Buildings, *Journal of Computing in Civil Engineering*, 16(2): 83–103.
- Hall, J.F.T.E., 1995. Nonstructural Damage, Northridge Earthquake of January 17, 1994 Reconnaissance Report, *Earthquake Spectra* Vol. 1: 453–513.
- Hoenderkamp, J.C.D. & Snijder, H.H., 2003. Preliminary Analysis of High-Rise Braced Frames with Facade Riggers, *Journal of Structural Engineering*, 129(5): 640–647.
- Kicinger, R. et al. 2005. Evolutionary Design of Steel Structures in Tall Buildings, *Journal of Computing in Civil Engineering*, 19(3): 223–238.

- Kuang, J.S. & Li, S., 2005. Interaction-Based Design Formulas for Transfer Beams: Box Foundation Analogy, *Practice Periodical on Structural Design and Construction*, 10(2): 127–132.
- Li, B., Duffield, CF & Hutchinson, GL (2007). Field Investigation Report – Analysis of Connection Properties, Department of Civil & Environmental Engineering Research Report, RR/Struc/11/2006, The University of Melbourne.
- Mahendran, M. & Moor, C., 1999. Three-Dimensional Modeling of Steel Portal Frame Buildings, *Journal of Structural Engineering*, 125(8): 870–878.
- Melchers, R.E., Editor 1990. Newcastle Earthquake Study, The Institution of Engineers, Australia.
- Naeim, F., 1999. Lessons learned from performance of non-structural components during the January 17, 1994 Northridge Earthquake – case studies of six instrumented multistory buildings, *Journal of Seismology and Earthquake Engineering*, Vol. 2(1): 47–57.
- Su, R.K.L., Chandler, A.M., Sheikh, M.N. & Lam, N.T.K., 2005. Influence of non-structural components on lateral stiffness of tall buildings, *The Structural Design of Tall and Special Buildings*, 14(2): 143–164.
- Sarma, K.C. & Adeli, H., 2005. Comparative Study of Optimum Designs of Steel High Rise Building Structures Using Allowable Stress Design and Load and Resistance Factor Design Codes, *Practice Periodical on Structural Design and Construction*, 10(1): 12–17.
- Sev, A., 2001. Integrating Architecture and Structural Form in Tall Steel Building Design, *CTBUH REVIEW*, Volume 1, No. 2, CTBUH.
- Stafford Smith, B. & Coull, A., 1991. Tall building structures: analysis and design. New York: Wiley.
- Wu, J.C., Lu, W.C. & Hsu, W.C., 2006. Implementation of a Feasible Control Design Process Incorporating Robustness Criteria for Wind-Excited High-Rise Buildings, *Journal of Structural Engineering* 132(1): 89–101.



FIELD INVESTIGATION REPORT

-- Analysis of Connection Properties
(DRAFT)

Research Report RR/ Struct/11/2006

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This report is part of an on going work of Bing Li's PhD research project in the University of Melbourne and in collaboration with the following industry partners and associates:

- Bovis Lend Lease Pty. Ltd.
- National Association of Women In Construction

The opinions and conclusions in this report are those of the authors and do not necessarily represent the views of any of the organizations involved in the study.

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Exclusive Summary

Dock 5 is a 32-storey high-rise residential building located in VIC Harbour, Melbourne, Australia. It is composed by a flat-slab reinforce concrete main apartment and a 9-storey reinforced attached apartment. This report introduced in detail the different types of structural and non-structural elements employed in this building, and analysed and classified the connections between these elements to identify the influence of different connections to the lateral performance of high-rise structures.



Dock 5, VIC Harbour, Melbourne

1. Introduction

Owing to the rapid increase of the slenderness ratio of high-rise buildings, lateral performance, the dominant factor in high-rise building design, shows more and more significance and complexity in the overall behaviour of buildings. There are great deals of different parameters such as element properties, material properties, loads conditions, and connection properties, etc. which contribute to the overall lateral performance of a building. Though each of these aspects can be the pilot influencing factor, contribution from connections of a building will be the focus of this report.

1.1 Review of Recent Research

In this report, connections of the case-study building have been divided into three groups:

- (1) Connections between structural elements
- (2) Connections between structural and non-structural elements
- (3) Connections of non-structural elements

1.1.1 Connections between Structural Elements

It is commonly recognized that the connections between structural elements, such as beam-to-column connections, are not perfectly rigid, and the properties of those semi-rigid connections have been investigated for several decades. Based on the difference of construction materials, connections between structural elements can be further categorized into bolted connection, welded connection, and reinforced concrete connection, etc. Bolted and welded connections are normally used in steel structure and composite structure. Most of studies about connections were developed within this area [1-35]. To identify the behaviour/ properties of beam-to-column connections are the most popular objectives of those research.

To check the effect of joints on the stability behaviour of steel structures, Masarira [7] investigated eight types of connections by both numerical analysis and finite element models. It was pointed that inaccuracy in assessment of the effect of joints on stability of structure frame appears in standards of most countries. Ignore those effects could be uneconomical. Gaps exist between current practice/ standards and the real behaviour of building connections,

while improvement methods were also presented when realizing those gaps [1, 2, 9, 10, 33] . Silvia et al. [2] and Bayo et al. [9] both developed simplified models based on the conventional analytical spring model for semi-rigid connections to diminish the limitation imposed by both β factor proposed in Eurocode 3 and the model itself. Bolted moment connections and connections reinforced with the lengthened flange rib were suggested by Yu et al. [1, 33] and Chen et al. [10] to be efficient and practical in improving structural performance.

Some detailed methods in analysing and predicting the property of connections have also been identified [3, 4, 30, 32, 34, 35], accompanying with findings of specific connection properties. When being applied to bending moment, different failure mechanisms of bolt connections between steel I beams such as web crushing, bolt failure and uni-axial bending failure have been detected in Olsen's research [27]. According to his analysis, with the increase of the endplate thickness while the same bolts, the moment bearing capacity of the connection increases. Zaharia et al [32] analysed the stiffness of joints in bolted connected cold-formed steel trusses by using a series experiments. The result emphasized that the joint deformability is mainly due to the bearing work of the bolts. Moreover, there will be only 2% difference for the ultimate load while 37% difference for the corresponding displacement when doing analysis considering both axial and rotational stiffness. Full-scale tests of steel-concrete composite connections have been conducted by Liew et al [35]. It identified that composite connection properties have close relationship with reinforcement ratio, steel element stiffening, and concrete encasement. Properties of connection with fillet welds and self-piecing riveted connection are also discussed by Kudzys [34] and Porcaro et al [3].

1.1.2 Connections between Structural and Non-structural Elements

As part of building, non-structural elements such as façade, infill walls, windows and doors, etc. have important functions in aesthetic, environmental, energy control aspects. Generally, they all have direct or indirect interactions with primary structure through connecting devices such as bolts and welds, etc. However, there are not so many researches done in analysing the properties of those connections so far as reviewed.

1.2 Requirements from Standards

According to Australian Standards of steel structures (AS 4100), following requirements of design of connection have been given.

1.2.1 When members subject to axial tension (Section 7):

- “When a connection is made by bolting or welding to all elements of the member cross-section, the member may be assumed to have a uniform stress distribution across the cross-section (Clause 7.3.1)”
- “When the ends of members are connected such that not all elements of the member cross-section attached to the support, then additional stresses resulting from shear lag or eccentricity are induced and should be accounted for in the design.(Clause 7.3.2)”
- The design requirements of members with pin connections are “intended to prevent tearing-through at the end of the eye-bar and dishing of the plate around the pin.(Clause 7.5)” These provisions are summarized in Figure 1

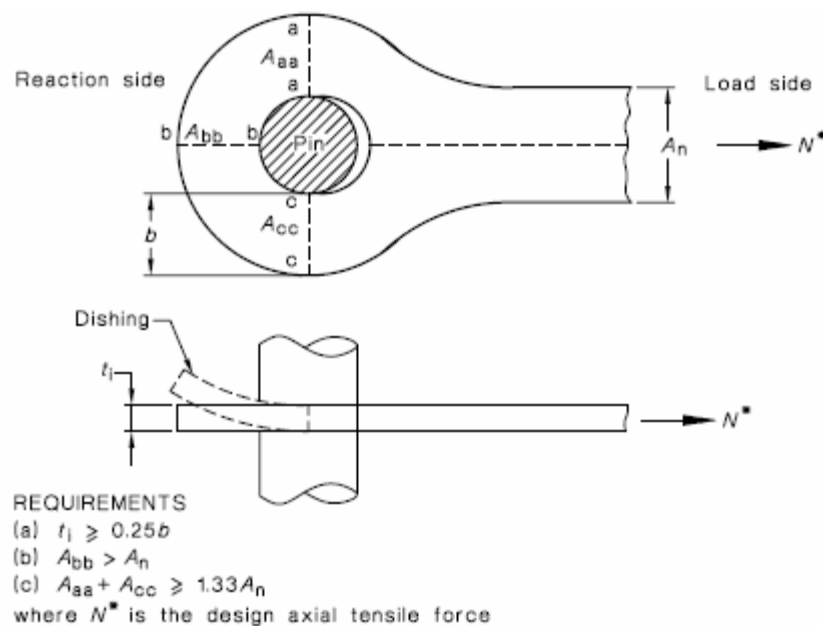


Figure 1. Pin connection for a single plate member (source: AS 4100 Supp1-1999)

1.2.2 Three forms of construction: rigid, semi-rigid, and simple (Clause 4.2).

- *“It is important to note that practical connections are neither fully rigid nor fully flexible...semi-rigid connection design demands a knowledge of the true moment-rotation behaviour of the connection to enable a frame analysis to be carried out, and to allow the design of the connection itself.*
- *Practical simple connections will transit some bending moment to the supporting members...Loss of rigidity in a rigid connection will cause a redistribution of bending moments in a frame.*
- *The rotation behaviour of practical simple connections is most commonly provided for by allowing one or more elements in the connection to deform appreciably...”*

1.2.3 Design of bolts, pin connections and welds (Section 9)

Details refer to Table 1.

Table 1. Design requirements on bolts, pin connections, and welds

	Bolt	Pin	Weld
Strength	<p>Shear</p> $f_{df} = 0.62 f_{df}$ $V_{fn} = 0.62 A_v f_{df}$ $V_{fx} = 0.62 A_v f_{df}$ <p>For bolted lap splice connections, a reduction factor k_r can be used</p> $k_r = 1.0$ $l_j \leq 15d_f$ $= 1.075 - \frac{l_j}{200d_f} \quad 15d_f < l_j \leq 65d_f$ $= 0.75 \quad l_j > 65d_f$	<p>The nominal shear capacity is based on a shear stress at failure of 62% of the yield stress of the pin material, as for a bolt subject to shear force</p>	<p>Fillet welds</p> $\sqrt{\left[\frac{v_n^{*2}}{k_w} + k_v \left(v_v^{*2} + v_{vt}^{*2} \right) \right]} = k_w (0.6 f_{tw} t)$ $k_v = 1.0 = \phi k_w v_w$ $k_w = 1.0$ <p>An alternative approach is to use a load-deformation method. Plug and slot welds</p> <p>Be used to transmit shear in a lap joint or to prevent the buckling of separation of the plates in a lap joint. The use is not extensive for structural applications.</p> <p><i>Diameter of the hole for a plug weld:</i> no less than the thickness of the part containing it plus 8mm. The diameter should not exceed either the minimum diameter plus 3mm or 2.25 times the thickness of the part, whichever is greater</p> <p><i>The minimum center to center spacing of plug welds :</i> 4 times the diameter of the hole</p> <p><i>Depth of the filling of plug welds:</i> in material 16mm or less should be equal to the thickness of the material. For over 16mm, should be at least one half of the thickness of material, but no less than 16mm.</p> <p><i>Length of the slot:</i> should not exceed 10 times the thickness of the part containing it.</p> <p><i>Width of the slot:</i> should not exceed either the minimum width plus 3mm, or 2.25 times the thickness of the part, whichever is the greatest.</p> <p><i>The ends of the slot:</i> semicircular or have corners round to a radius not less than the thickness of the part containing it. Except those ends extend to the edge of the part.</p> <p><i>The minimum spacing of lines of slot welds</i> in a direction transverse to their length should be 4 times the width of the slot.</p> <p><i>The minimum center to center spacing</i> in a longitudinal direction on any line should be 2 times the length of the slot.</p>
	<p>Tension</p> $N_{df} = A_v f_{df}$		
Combination	<ul style="list-style-type: none"> • Elliptical interaction relationship • The nominal tension capacity and the nominal shear capacity used in the denominators of the interaction equation are the respective nominal capacities of the bolt under the separate individual loads, with the nominal shear capacity being dependent upon the locations of the shear planes, s for a bolt subject to shear force alone. 		
Bearing	<p>Is not considered as a possible failure mode</p>	<p>The relatively low failure stress of 1.4 times the yield stress of the pin material reflects the critical nature of this load on a single pin. The factor k_p of 0.5 for a pin that allows rotation reflects the fact that continual movement of the pin plates around the pin circumference creates a wearing effect.</p> <p>A pin is treated as a compact member, subject only to plastic yielding.</p>	

		<p>Long end distance in the direction of applied loads $V_b = 3.2 f_{up} d_f t_p$ Short end distance: plate-tearout failure $V_b = a_e t_p f_{up}$</p>	Use definition as bolt	
Ply in bearing		<p>Elastic analysis:</p> $V_n^* = \frac{r_n - V_{df}^*}{r_{max}}$ $x_e = \frac{-(\sum y_n^2 + \sum x_n^2)}{n_b e}$ $y_e = 0$ $V_{df}^* = \frac{-V^* r_{max}}{\sum (x_n - x_e)}$ $V_{bn}^* = \frac{V^*}{n_b}$ $V_{bc}^* = \frac{(V^* e) r_n}{\sum x_n^2 + \sum y_n^2}$ <p>Plastic analysis Assume all bolts not at the centre of rotation are deformed sufficiently to become fully plastic and all transmit the same force and failure.</p> <p>Other methods</p>		<ul style="list-style-type: none"> In-plane loading: <ul style="list-style-type: none"> Linear elastic method Alternative method Out-of-plane loading: <ul style="list-style-type: none"> Methods identified in other documents
Serviceability		<ul style="list-style-type: none"> <i>The maximum amount of slip on connections (not classified as slip-critical): 2-3 mm.</i> <i>Considerable variation in both the initial bolt tension N_i and the function of the surface condition of the interfaces u depend on the bolt grade and the method of installation.</i> 		
Design details		<ul style="list-style-type: none"> Minimum pitch: 2.5 bolt diameters Minimum edge distance: be controlled by end plate tearout Maximum pitch: based on successful past practice Maximum edge distance: based on successful past practice, also intended to prevent any potential curling up of plate edges 		

Nomenclature:

ae	<i>long end distance</i>
A_s	<i>tensile stress area</i>
df	<i>bolt diameter, has been chosen as 20 mm</i>
tp	<i>thickness of the ply</i>
f_{pu}	<i>nominal bearing strength of the ply</i>
f_{vf}	<i>average shear strength of the bolt</i>
f_{uf}	<i>tensile strength of the bolt</i>
r_n	<i>distance from centre of rotation to the applied force</i>
N_{tf}	<i>the tensile capacity of a bolt</i>
V^*	<i>design action</i>
V_b	<i>nominal bearing capacity of a ply</i>
V_{bc}	<i>the force on each bolt under the design action couple V^*e</i>
V_{bv}^*	<i>the force on each bolt when design action act at the bolt group centriod</i>
V_{bu}	<i>the ultimate bearing capacity of a ply</i>
V_n^*	<i>force on any bolt</i>
v_n^*	<i>design force per unit length of weld normal to the plane of the fillet weld throat</i>
v_l^*	<i>design shear force per unit length of weld longitudinal to the plane of the fillet weld throat</i>
v_t^*	<i>design shear force per unit length of weld transverse to the plane of the fillet weld throat</i>
V_{of}^*	<i>force on the bolt furthest</i>
V_f	<i>nominal shear capacity of a single bolt</i>
V_{fn}	<i>nominal shear capacity of a single bolt for threads intercepting one shear plane</i>
V_{fx}	<i>nominal shear capacity of a single bolt for a plain shank intercepting one shear plane</i>
(x_e, y_e)	<i>centre of the rotation</i>

1.3 Objectives of This Report

Based on the above literature, it is obvious that the role played by structure connections is critical to the structure behaviour. From rigid consideration to the semi-rigid modification using bi-linear spring models, connections between structural elements such as beam-column connections have already been widely developed and analysed. However, even there are some evidence illustrated that non-structural components will influence the lateral performance of high-rise buildings dramatically, seldom investigations to the interface between building structural and non-structural elements have been done. This report is therefore focusing on this special interface, and will be completed by achieving following 3 main objectives:

- To identify different type of structure intersections of Dock 5
- To clarify the properties of different types of connections
- To quantify the influence from structure connections to the overall lateral performance

General introductions of case study building and properties of key elements will be used as the background. Detailed field investigations results about different types of connections are provided to be the analysing support. Finally simplified Finite Element model will be used to identify the influence of connections to the structure performance.

2. Dock 5 General Information

The case study building involved in this analysis is an on going project named Dock 5 in VIC Harbour, Melbourne, Australia. This building is designed and constructed as a typical reinforced concrete structure with structural and non-structural facades for residential purpose.

There are mainly two parts composing of Dock 5 building. The primary 32-storey building and a 9-storey apartment connected rigidly to the main building. The building has irregular floor planes of polygons different from storey to storey. It has two concrete shear cores, and combined structural form of framed and flat slab structure. In a typical floor plane, it can be easily seen that the plane area is roughly 4000m^2 , with reinforced concrete beams and columns as structural frame, together with reinforced concrete floor slabs and reinforced concrete shear walls to be the primary structure.

Major materials used in this project are reinforced concrete, precast concrete and steel. There

are also masonry and timber, as well as glass involved, which are not belonging to the main stream. Depending on different elements, concrete used in this building has the strength from 32 to 80 Mpa, with re-bar diameters of N12 to N36.

3. Details of Elements

3.1 Structural Elements

In this report the structural elements means the elements compose of primary structure. It includes beams, columns, floor slabs, and shear walls. All the element details follow the requirements of AS/NZS 3600.

3.1.1 Floor Slabs

Floor slabs are the main horizontal elements that transmit both the live loads and the dead loads to the vertical framing supports of a structure [36]. In this building, reinforced concrete (RC) is used for most of floor slabs. The concrete strength grade is 32 Mpa and the diameters of rebar vary from N12 to N36. Both top and bottom reinforcement are applied. The typical thickness of floor slabs is 190mm. Some prestressed (P/T) slabs are also used with different concrete strength grade of 40 Mpa.

3.1.2 Beams

Beams are the structural elements that transmit the tributary loads from floor slabs to vertical supporting columns [36]. Beams employed in Dock 5 building basically can be divided into two types: general beam and band beam. They are cast monolithically with the slabs and perform as T-beam or L-beam. The same as floor slabs, reinforced concrete are used for all the beams, with concrete strength grade of 32 Mpa and rebar diameter from N16 to N36. Both top and bottom reinforcement are involved in the two types of beams.

3.1.3 Columns

Columns are the vertical elements which support the structural floor system and transmit axial compressive loads, with or without moments [36, 37]. In this building, most of the columns have rectangular cross section. All of them are reinforced concrete columns with concrete strength grade of 80-32 Mpa and rebar diameters from N16 to N36.

3.1.4 Shear Walls

Shear walls are the structural concrete walls to resist lateral loads. The thickness of the reinforced concrete shear wall in Dock 5 varies from 150 mm to 400 mm. Strength grade of 80-32 Mpa concrete is used with both horizontal and vertical reinforcement by rebar which diameters are from N16 to N36.

Table 2. Details of structural elements

Element	Cross section mm ²	Thickness mm	Re-bar Diameter mm	Concrete grade MPa
Beam	T- L-		N16~N36	32
Column	Rectangular		N16~N36	80~32
Floor slab		190	N12~N36	32~40
Shear wall		150~400	N16~N36	

3.2 Non-structural Elements

The non-structural elements mentioned in this report are those which are designed as non-load bearing elements. They will include infill walls, façade panels, doors, windows, etc.

3.2.1 Infill Walls

Infill walls in Dock 5 are precast concrete walls and brick infill walls, such as stairwells. These walls are not designed as major load bearing walls, so they are generally thinner than those RC walls, around 150 mm to 200 mm thick. The concrete strength grade is 40 Mpa, with reinforcement as required. All the details of the precast are in accordance with AS/ NZS 3600, 3850, 1554.

3.2.2 Façade

The non-structural glass façade panels have important functions of protecting the building from high noise and wind levels. Dock 5 employed the double-skin modern façade technique by using glass with different colours and aluminium frames.

3.2.3 Doors and Windows

Doors and its frames in this project are made of timber or metal. Windows are formed by single-or double-skin glass together with metal frames. Owing to its residential purpose, most of doors and windows in this building have standard dimensions and properties.

4. Connections between Elements

Generally, connections in the building can be divided into three categories according to different types of elements (Table 2-4):

- Connections between structural elements (Table 2). This means the connection among beams, columns, walls and floor slabs;
- Connections between structural and non-structural elements (Table 3). This type of connections focuses on the interface between structural elements and non-structural elements, that is, how the non-structural elements been connected to the primary structure.
- Connection within the non-structural elements (Table 4). These are the details of the connection between different parts of non-structural components.

As identified in the previous section, the purpose of this research is to investigate the influence of different connections to the overall performance of high-rise buildings under serviceability loads. In this case, the probability of the appearance of plastic hinges at reinforced connection points and the cracking problem of concrete in tension could be ignored. Consequently, connections between reinforced concrete structural elements will be considered as rigid connections.



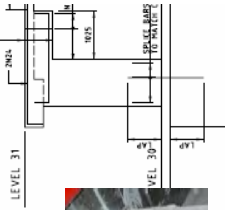

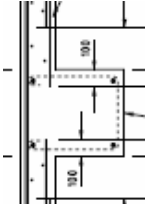

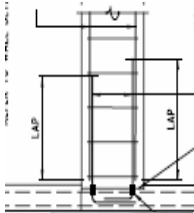
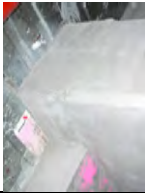
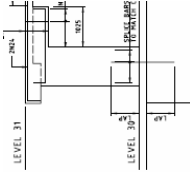
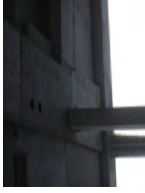
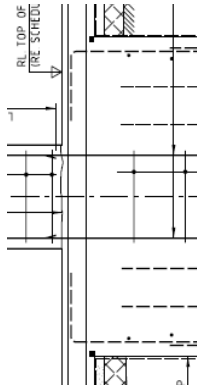

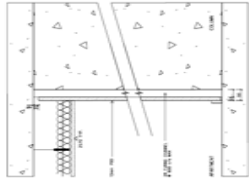
Since structural elements in this building are mainly reinforced concrete elements while the non-structural components such as façade assembly and windows are glass or timber panels with metal frames, structural and non-structural elements of Dock 5 are connected mainly by fasteners (e.g. nails or screws/ bolts). In this scenario, the connecting property becomes an important index of the structural performance. It can directly influence the load transfer between structural and non-structural elements. Different connection properties can lead to

different loading and failure mechanism of the structure. In this circumstance, it is important that this type of connections be analysed and examined carefully.

The inner connections of different non-structural components are metal-to-metal connections, metal-to-timber connection, or metal-to-glass connection, etc. Metals are always connected together by bolts or weld, so do timbers. Even though such connections will have great influence to the stiffness contribution of non-structural components, regulated by the detailed inductions and operation standards from manufacture industry, they always become the influencing factor of the stiffness of single non-structural components which is one of the properties provided by the manufactory.

Thus, referring to the purpose of this report, connections between structural and non-structural components are the focuses.

Table 3. Connections between structural elements

	<p>Beam</p> <ul style="list-style-type: none"> • Double reinforced and continuous • Beam to beam connections are made through crossing rebar and concrete coat. 	<p>Column</p> <ul style="list-style-type: none"> • Rigid connection • Beam to column connections are made by high tensile bars  	<p>Slab</p> <ul style="list-style-type: none"> • Rigid connection  	<p>Wall</p> <ul style="list-style-type: none"> • Semi-rigid connection  
<p>Column</p>	<ul style="list-style-type: none"> • Rigid connection • Beam to column connections are made by high tensile bars  	<ul style="list-style-type: none"> • Columns are always under compression and are classified as short or long and slender columns • Column to column connection made by dowel bars 	<ul style="list-style-type: none"> • Rigid connection  	<ul style="list-style-type: none"> • Rigid connection  

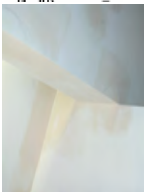
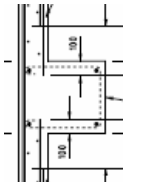

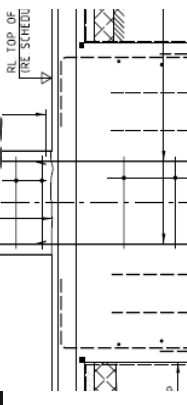

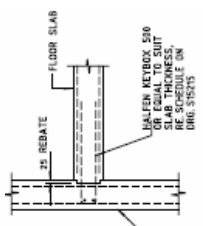

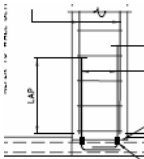


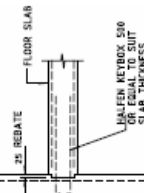



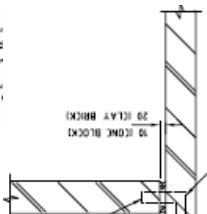
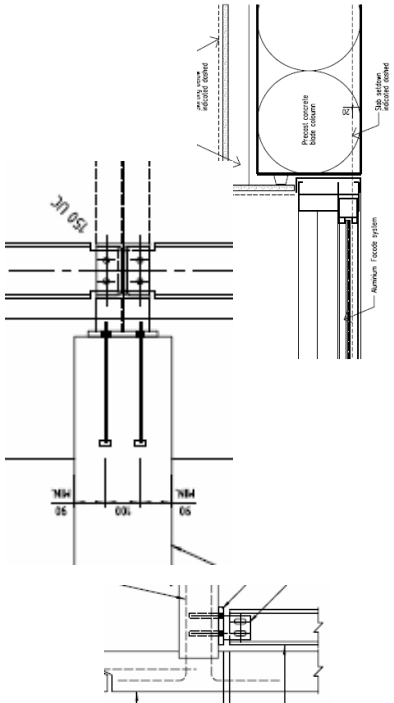


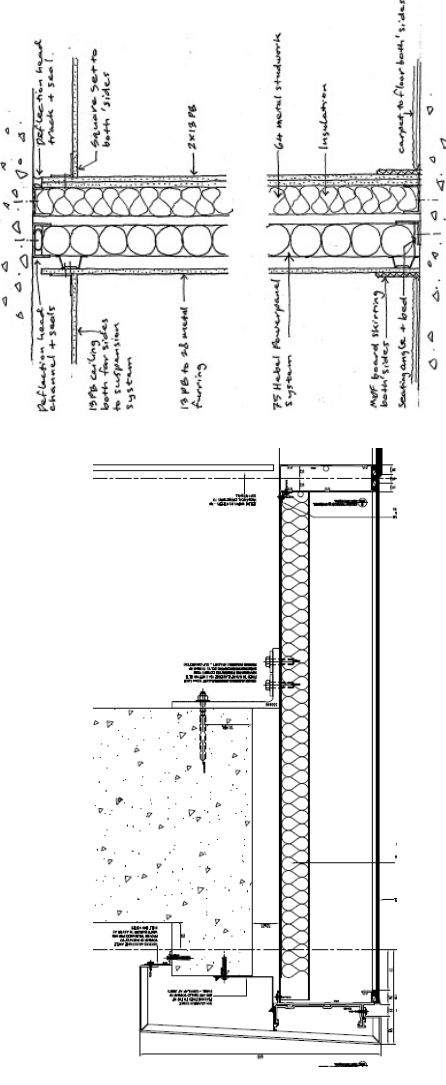
	<p>Beam</p> <ul style="list-style-type: none"> • Rigid connection  	<p>Column</p> <ul style="list-style-type: none"> • Rigid connection  	<p>Slab</p> <ul style="list-style-type: none"> • One-way, two-way Double Reinforced 	<p>Wall</p> <ul style="list-style-type: none"> • Semi-rigid connection  
<p>Slab</p>	<ul style="list-style-type: none"> • Semi-rigid connection  	<ul style="list-style-type: none"> • Rigid connection 	<ul style="list-style-type: none"> • Semi-rigid connection  	<ul style="list-style-type: none"> • Rigid, semi-rigid, and pin connection    

Table 4. Connections between structural and non-structural elements

	<p style="text-align: center;">Structural Components</p>
	<p style="text-align: center;">Windows & Doors</p>
<p style="text-align: center;">Connection Type A</p>	

	<p style="text-align: center;">Structural Components</p>	
<p style="text-align: center;">Façade</p>	<p style="text-align: center;">Connection Type B</p>   	<p style="text-align: center;">Partitions Infill wall</p> 

Structural Components	
Curtain wall	<p>Connection Type D</p> <p style="text-align: right;">FOR CONSTRUCTION</p>
Stairs	<p>Connection Type E</p> <p style="text-align: center;">SECTION A</p> <p style="text-align: center;">TYPICAL LANDING FOR STAIR TYPE S1176 & S1175</p>

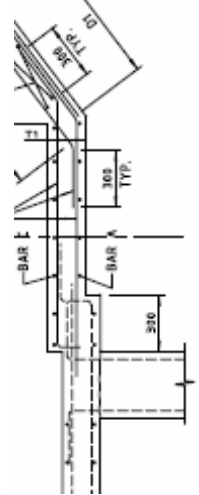
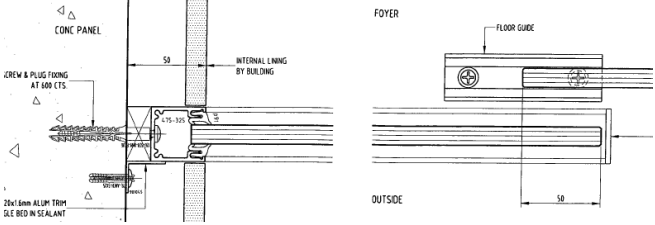
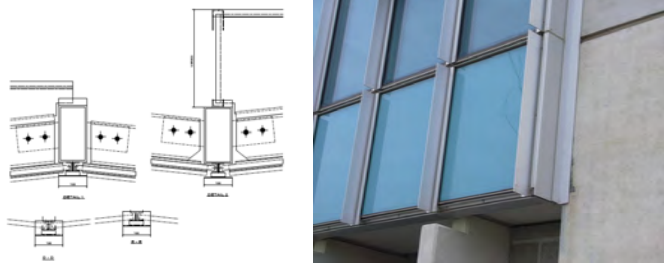
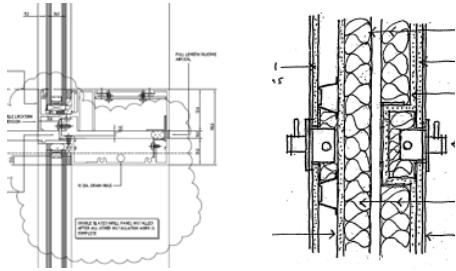
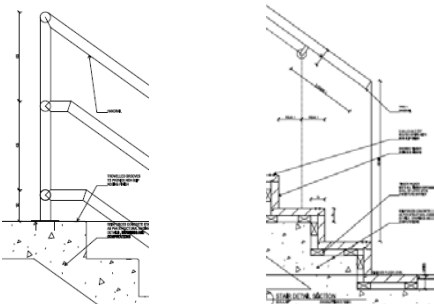


Table 5. Connections between non-structural components

	Inter connection
Windows	<p style="text-align: center;">Connection Type I</p> 
Façade	<p style="text-align: center;">Connection Type II</p> 
Partitions Infill walls	<p style="text-align: center;">Connection Type III</p> 
Stairs	<p style="text-align: center;">Connection Type V</p> 

5. Sensitive Model Analysis

From the above section, the connections need to be analysed in detail are the connections between structural and non-structural components. By analysing the above tables and design details of the building, three types of connections need to be further investigated.

- Composite connections. This typical expression of connections is as shown in Figure 2. Steel supporting panels of non-structural elements are fastened by bolts, nails, or screws to concrete structural components.

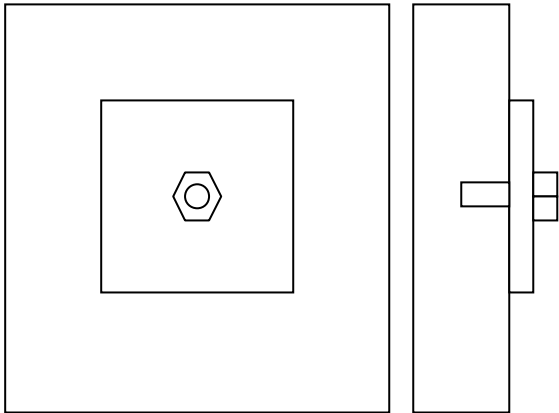


Figure 2. Composite connection

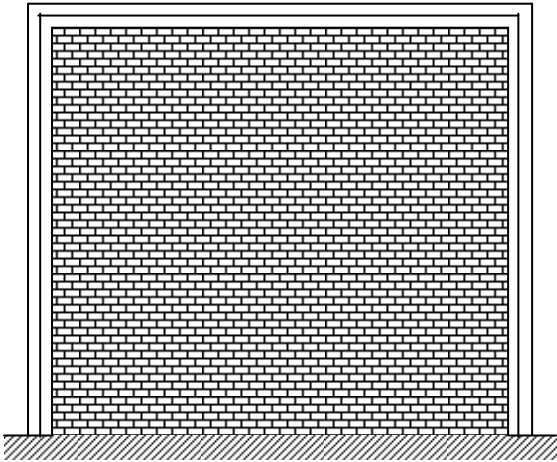


Figure 3. Infill wall

- Metal connections. Some of the non-structural elements are connected to steel beams through bolts or screws, while some may be connected to the main structure by weld.

- Infill Walls. Infill wall such as stairwell is a typical type of non-structural elements. The simplified illustration of the connection between main structure and the infill wall is shown as Figure 3.

FE models have been developed to analyse the structure performance. Prior to this study, a lot of research has been done in finite element analysis of building behaviour [38-42]. It is reasonable that the reflection of structure performance from 3-D model will be more accurate than that of 2-D model [38-42]. So in this study, 3-D analysis was developed.

5.1 Software

The commercial FE analysis package ANSYS 10.0 (University Introduction Version) has been employed as analytical tools to simulating and calculating the F-D relations of different structure configuration under quasi-static loading conditions. There are two main advantages to choose this software package

- Various of elements can be used to define different structure systems
- Complex enough for finite element analysis of non-linear large displacement performance

Finite element models have been built to analyze the influence to the overall structural stiffness caused by properties of these connections.

5.2 Assumptions

In this section, failure mechanisms of different types of connections as well as some assumptions about material properties, structural details, and boundary conditions will be identified.

5.2.1 *Failure mechanisms of different types of connections*

In reviewing current research, failure mechanisms of composite connection, mental connection and infill walls can be explained as follows.

- *Composite connections*

Bolts are used to fasten the steel ply and concrete structure. Based on Kabche et al [14], the theoretical failure mechanism of composite joints will be:

- Concrete failure. This failure mechanism is due to the failure of concrete bearing capacity. In this case, the connection might fail to carry the load because of concrete cracking or even destroy of surrounding concrete. Figure 4.

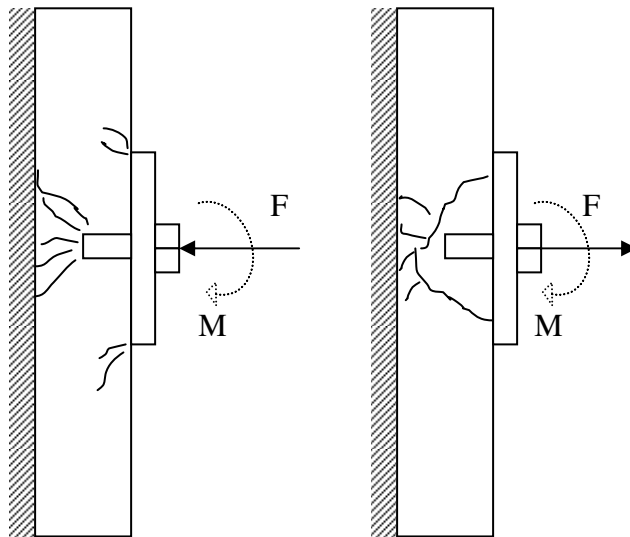


Figure 4. Concrete failure in composite connection

- Steel ply failure. The steel ply failure will mainly be induced by the failure of the hole. When subject to shear force, if the ply material is not strong enough to bear the shear strength distributed, or if the hole on the ply has initial defections such as sharp angle etc. which is the weakest point of the ply, buckling and failure of the steel are easy to happen. Figure 5.
- Bolt failure. Bolt failure normally happened because of lack of bearing capacity in material strength. Similar to ply failure, if the bolt material is not strong enough, when subject to shear forces or bending moment, bolt failure will firstly happened.

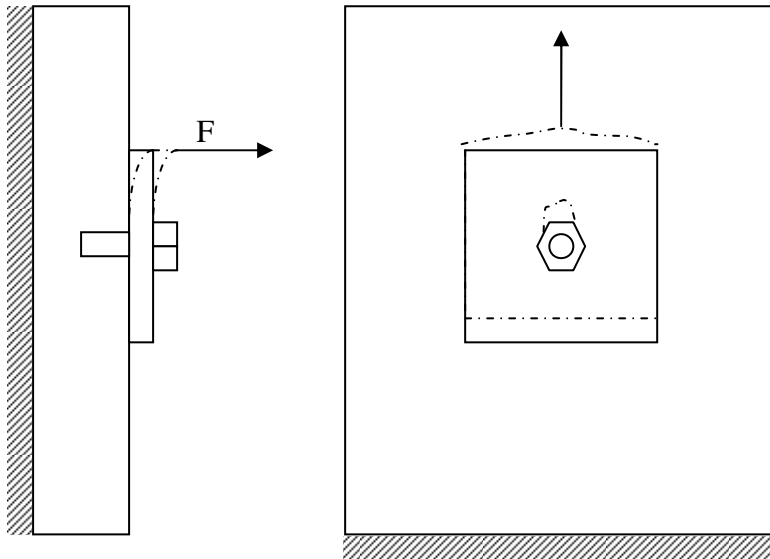


Figure 5. Steel ply failure in composite connection

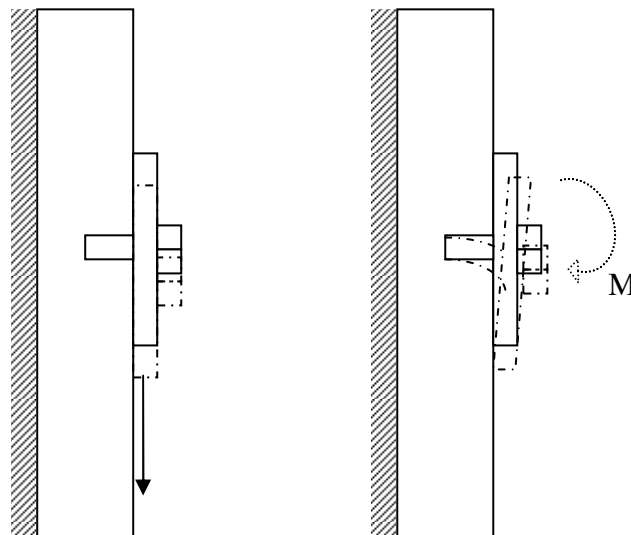


Figure 6. Bolt failure in composite connection

- *Steel Connections*

Here steel connection means steel elements connected by bolts/screws. According to Casafont et al [16], Rodrigues et al [23] and Olsen [27], failure mechanisms of this type of connections will include:

- Ply failure. Similar to composite connections, ply failure will mainly be tearing failure. Figure 7.

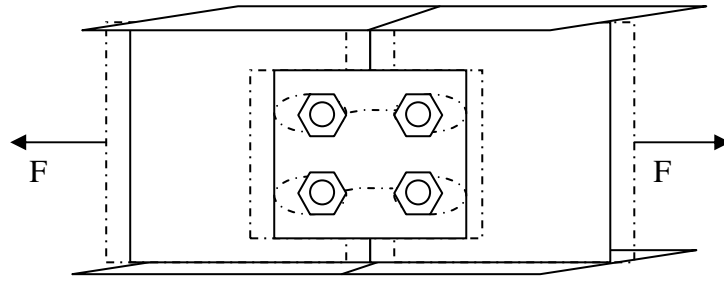


Figure 7. Ply failure in steel connection

- Bolt failure. Owing to the material property or the installation inaccuracy, bolt failure will happen when the shear force or bending moment exceed the bearing capacity of bolts. Figure 8.

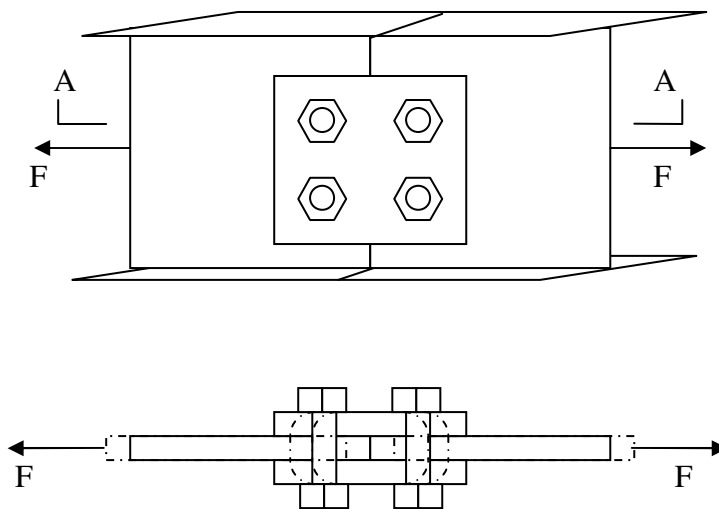


Figure 8. Bolt failure in steel connection

- Element failure. The element failure will happen when the material strength is not enough or element dimensions can't reach the bearing requirements, etc. Usually those failures occur under ultimate limit state loads.

- *Infill Walls*

For the infill walls, we assume that failure mechanism will be taken into account at the time when the infill wall starts to bear loads. Figure 9.

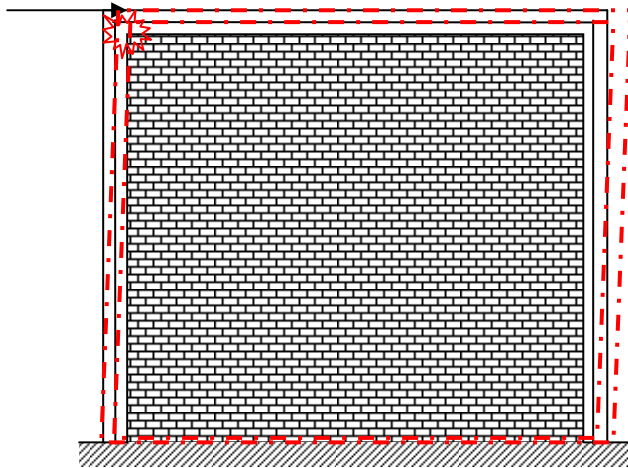


Figure 9. Infill wall

5.2.2 Assumptions

The assumptions listed below are made to simplify the analysis and to define the scope of the modelling procedure.

- *Element Configuration.*

The structural elements which are involved in this study will be beam, column, and structural wall. To make the analysis simpler and general, beam and columns will have unified rectangular cross-section areas, and structural walls will have unified thicknesses. Non-structural elements under investigation are infill walls and façades. The connections are bolted connections and infill walls. Spring properties will be used to bolted connections so that the contribution from connection stiffness to the overall structure performance will be quantified. Distance between infill walls and structure frame will be included when simulating the relationship of them.

- *Material assumptions*

Since this report is to identify the contribution from building connections to the overall structure performance, the comparison analysis won't be influenced by the material properties of both structural and non-structural components. So, in this report, the structural elements are all assumed to be concrete elements without reinforcement. The infill walls are also pure concrete walls, while the properties of façade panels are represented by very thin steel panels (to achieve the linear elastic property). Assume the connections between structural elements

and façade panels are springs, with steel properties. In this case, changing of connection properties can be expressed clearly by changing of spring stiffness.

- *Various of connections*

Based on the investigation in above sections, connections between structural and non-structural elements have highest priority in analysis. When taking the reality of this case study building into consideration, the bolt connection between structure frame and façade panels and the connection between infill walls and structure frame are the typical two connection types. Thus, in the modelling process, the connection influence from those types will be analysed in detail.

- *Details*

3-D FE analysis of a complex high-rise building carried by ANSYS has high requirements to computer hardware settings. It will become time and memory consuming at the average setting level of computers. To overcome this difficulty and make savings for computing time and memory, simplifications of the models are made. Assume:

- the building frame has been simplified to regular shape
- frame elements such as beams and columns have same cross-section properties
- there is no reinforcement of concrete material
- there is no variation of concrete grades

Since the purpose of this study is to identify the contribution from different building connections to the lateral performance of the structure, so long as the structure frame and non-structure details in different models keeps the same, there won't be any influence on the comparison analysis.

5.3 Modelling Approach

5.3.1 Material properties

Concrete and steel are the two types of material used in the models. Concrete was used for structure frame and infill walls (Figure 10), while steel was for connections and non-structural façades (Table 6).

Table 6. Steel properties used in models

	Yield stress	Young's modulus	Poisson's ratio	Tangent modulus	Density
	MPa	MPa		MPa	kg/m ³
Steel	640	2×10 ¹¹	0.299	2×10 ¹⁰	7850

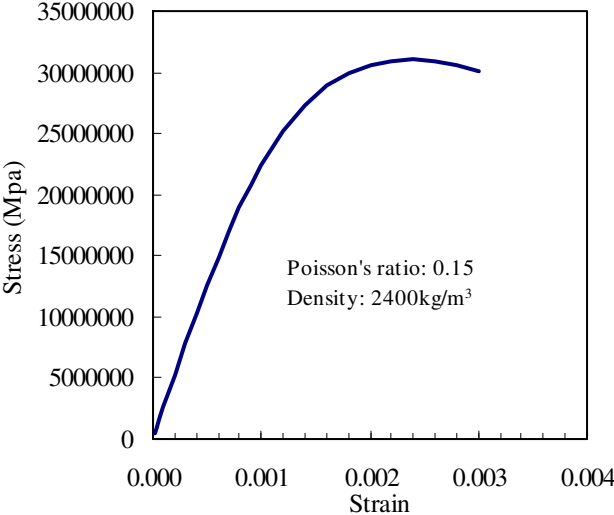


Figure 10. Concrete properties used in modeling

5.3.2 Element properties

Beam element, shell element, and spring element are used to simulate structure frame, façade, infill walls, and connections respectively. The cross section of beam element is a 0.8m×0.8m square. The shell element for infill walls is 0.15m thick while the shell element for façade panels is 0.02m. The k values of 0.1, 10, and 100N/m have been evaluated to the connections between frame and façade panels, in order to identify the influence from the stiffness of connections to the structure performance.

5.4 Model description

A frame which is 60m×60m in floor plan and 3m in storey height has been developed (Figure 11 (a)). The distances between frame and infill walls, as well as between frame and façade panels are as show in Figure 11 (b) and (c).

5.5 Scenario

In clearly identifying the influence caused by connection properties to building lateral performance, following model scenarios have been proposed to subject to the quasi-static lateral top loading (Figure 12).

- Frame only
- Frame + façade panels with k value vary from 0.1 to 100N/m
- Fame + infill walls with both total and partly bottom-constrained conditions

Comparison analysis will be conducted when the models built according to the above scenarios were computed by applying certain lateral loads. The deformation capacity or stiffness of building systems will be compared and analysis.

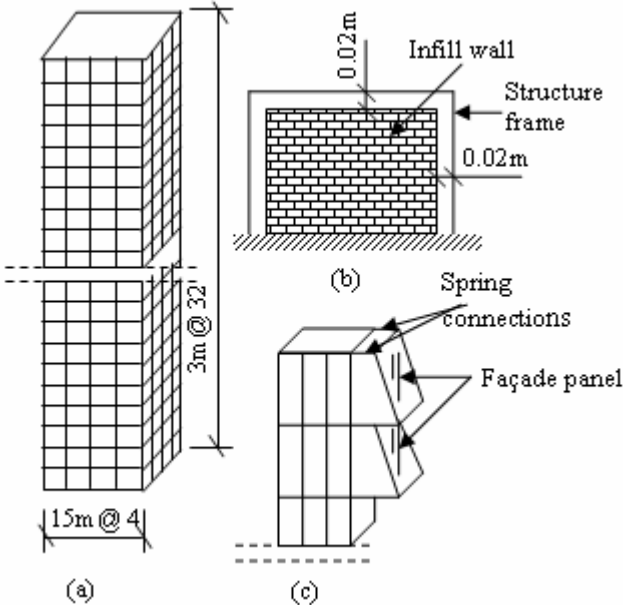


Figure 11. Model details

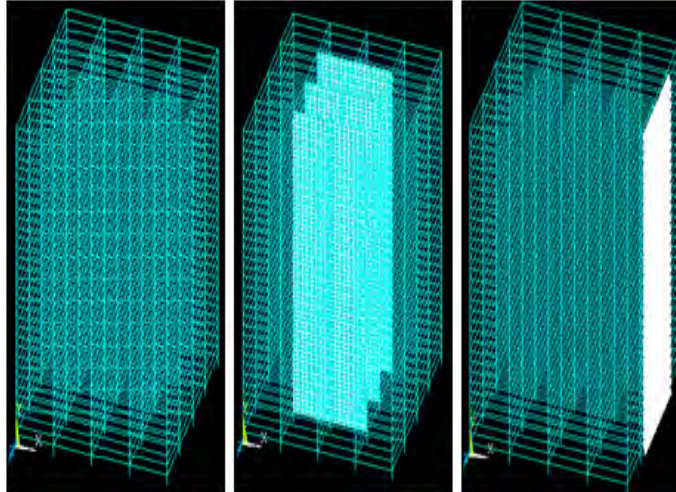


Figure 12. Models for different scenarios

5.6 Boundary and loading conditions

In all the models, total 6 degree of freedom of the frame base have been constrained, which means the frame of the structure is just like a cantilever. Displacements and rotations along x, y, z directions at the bottom of infill walls were coupled to the same storey of frame. To achieve the quasi-static loading condition, lateral loads along x direction at the top of building were applied gradually in small load steps.

5.7 Discussion

Results from the above models are plotted in Figure 13 and Figure 14. Figure 13 shows the comparison of different force-displacement (F-D) relations of frame-façade system with different value of connection stiffness ($k = 0.1, 10, 100\text{N/m}$). From the figure, when k equals to 0.1, 10, 100N/m respectively, the F-D relations are all overlapped with that of the frame only F-D relation, with the same slop of approximately 41kN/mm. That means the stiffness of the frame-façade system won't change according to the variation of connection stiffness.

However, when the external lateral load was scaled to a value big enough to make the FE solutions failure to get converged at some loading step, which means the software will consider the model has been destroyed at this stage, it can be observed that the load bearing capacity of the system increased with the increasing of connection stiffness. When k equals to 0.1N/m, the maximum loading to the building system could only be around 2000kN. When k rose up to 10N/m, the maximum load also increased, up to more than 200000kN. The same

trend happened when k was increased to 100N/m, again, the maximum load the system can bear reached to approximately 26000kN. This tendency reflected that the deformation capacity of the building system increases in direct proportion to the increment of connection stiffness. This phenomenon can be understood when plotting the failure deformation shape of the system. From the deformed shape, it can be identified that damages in models with different connection stiffness are all happened because of large deformation of façade panels caused by connection failure. Hence, interactions between structure frame and façade panels are expressed and enhanced by their connections. If those connections can be well improved, the load bearing capacity of the building system can also become higher.

Figure 14 shows when the bottoms of infill walls are fully constrained to the structure frame, the stiffness of building system will be improved dramatically. Under the same loading level, for instance, 20000kN, the top corner lateral displacements of the building systems with and without the totally constrained infill walls will vary from 75mm to 475mm approximately. Moreover, if the infill walls are partly constrained to the frame, the top displacement of buildings will reach to 145mm under same loading conditions, which means that the constrain conditions of infill walls can cause direct influence to the overall stiffness of structures. When constrain properties were enhanced, the stiffness of structures will increase accordingly.

5.8 Conclusions about modelling

From the discussion section, there are some findings from the model, which can lead to several conclusions:

- infill walls which are totally bottom-constrained to the structure frame has significant contribution to the structure stiffness. It can improve structure stiffness up to more than 5 times of the stiffness of frame-only structure;
- infill walls which are partly bottom-constrained to the structure frame has more than 200% stiffness contributions to the overall structures;
- when constrain properties of infill walls were enhanced, the stiffness of structures will increase accordingly;
- the stiffness of connections between façade panels and structure frame may not have great influence to the overall system stiffness

- the stiffness of connections between façade panels and structure frame has significant influence to the system load bearing capacity. The load bearing capacity of system will increase in direct proportion to the increment of connection stiffness;

The above points give sufficient evidence that the contributions from different types of building connections to the lateral performance of building system are significant enough to be analysed and considered in the design and construction process of high-rise buildings.

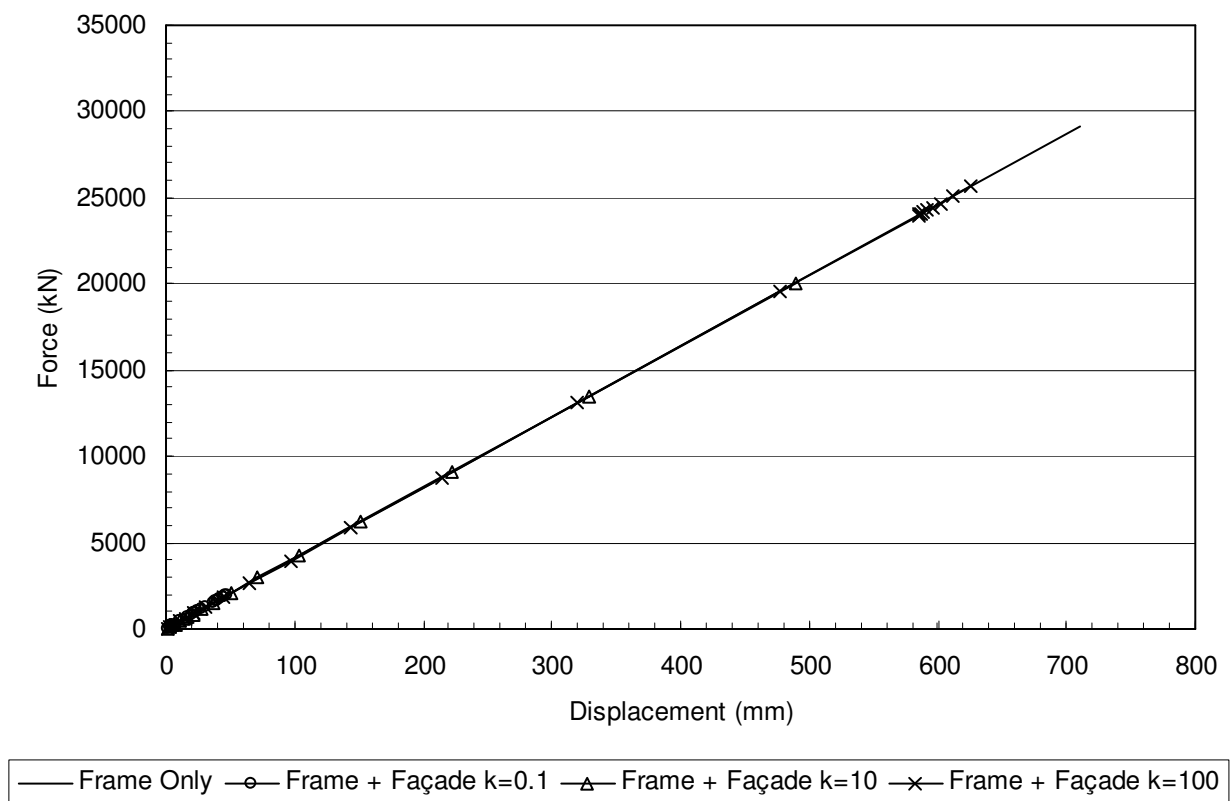


Figure 13. Comparison of building F-D relations under different connection properties between facade and structure frame

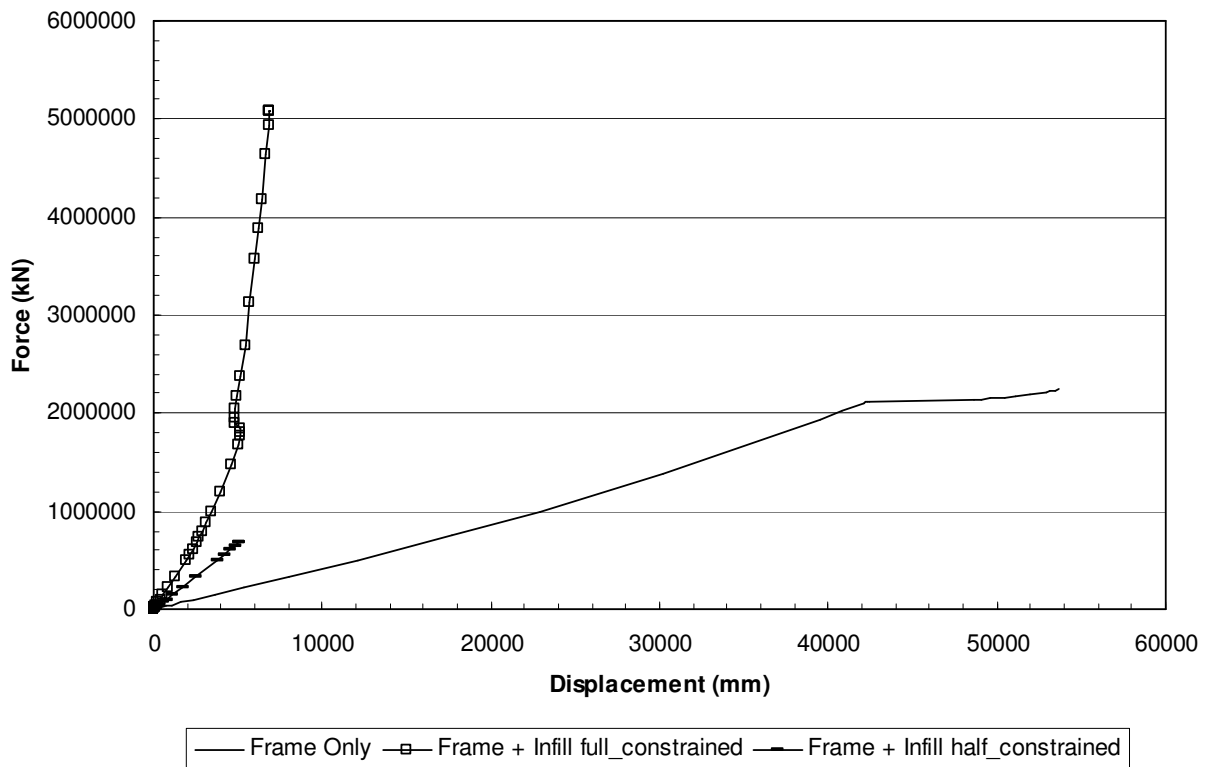


Figure 14. Comparison of building F-D relations under different connection properties between infill walls and structure frame

6. Conclusion

This report analysed different building connections by both field investigation and finite element modelling approach. The previous work on different connection behaviours have been reviewed and compared with requirements from Australia design standards (AS/ NZS 4100). A case study building was introduced and investigated focusing on different types of elements and connections. Based on the investigation, finite element models were built up to analyse in detail the influence from different connection properties to the lateral performance of the overall building system. Comparison analysis was conducted on the F-D relations drawn from different finite element models and conclusions were made according to the comparison.

Typical structural elements involved in the case study building are beam, column, wall, floor slab, etc. Critical non-structural components include façade panel, infill wall, door and windows, etc. When concerning on the connections and different element types, there will be connection between structural elements, connection between structural and non-structural

elements, and connection between non-structural elements. According to material difference of connecting elements, there will be bolted connection, weld connection, and concrete connections (connected rebar and concrete) etc. Detailed analysis on those different connections have been conducted by previous scholars, failure mechanisms of each type were identified.

Finite element models were developed by using commercial package ANSYS 10.0. Spring properties were assigned to connections between façade panels and structure frame after consideration of the real building situation and analysis purpose. Quasi-static loading approach was used when analysing the lateral performance of buildings system under different connection types and properties.

By comparing F-D relations of building systems with different connection properties, following conclusions can be drawn:

- infill walls which are totally bottom-constrained to the structure frame has significant contribution to the structure stiffness. It can improve structure stiffness up to more than 5.3 times of the frame-only structures;
- infill walls which are partly bottom-constrained to the structure frame has more than 200% stiffness contributions to the overall structures;
- when constrain properties of infill walls were enhanced, the stiffness of structures will increase accordingly;
- the stiffness of connections between façade panels and structure frame may not have great influence to the overall system stiffness;
- the stiffness of connections between façade panels and structure frame has significant influence to the system load bearing capacity. The load bearing capacity of system will increase in direct proportion to the increment of connection stiffness.

The field investigation and finite element modelling analysis illustrated that the role played by connections in a building is far more important than we considered to the structure lateral performance. The stiffness and load bearing capacity can be enhanced by increasing the connection stiffness, and vice versa. Therefore, it is necessary to pay special attention to the design and construction process on connections and interactions of high-rise building system.

7. Future Work

Through the field investigation and connection analysis, following tasks have been identified for the further development of this part of the project:

- Experimental analysis of different types of connections
- Detailed modelling of the connections
- Modification of connection and element material properties
- Analysis of other influential factors such as element dimensions and loading conditions
- Verification of the experimental and analytical analysis

Acknowledgement

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Reference

1. Yu, W.K., K.F. Chung, and M.F. Wong, *Analysis of bolted moment connections in cold-formed steel beam-column sub-frames*. Journal of Constructional Steel Research, 2005. **61**(9): p. 1332-1352.
2. Simoes da Silva, L. and A.M. Girao Coelho, *An analytical evaluation of the response of steel joints under bending and axial force*. Computers & Structures, 2001. **79**(8): p. 873-881.
3. Porcaro, R., et al., *The behaviour of a self-piercing riveted connection under quasi-static loading conditions*. International Journal of Solids and Structures, 2006. **43**(17): p. 5110-5131.
4. Dissanayake, U.I., J.B. Davison, and I.W. Burgess, *Composite beam behaviour in braced frames*. Journal of Constructional Steel Research, 1999. **49**(3): p. 271-289.
5. Cabrero, J.M. and E. Bayo, *Development of practical design methods for steel structures with semi-rigid connections*. Engineering Structures, 2005. **27**(8): p. 1125-1137.
6. Ahmed, B. and D.A. Nethercot, *Effect of column axial load on composite connection behaviour*. Engineering Structures, 1998. **20**(1-2): p. 113-128.
7. Masarira, A., *The effect of joints on the stability behaviour of steel frame beams*. Journal of Constructional Steel Research, 2002. **58**(10): p. 1375-1390.
8. Raftoyiannis, I.G., *The effect of semi-rigid joints and an elastic bracing system on the buckling load of simple rectangular steel frames*. Journal of Constructional Steel Research, 2005. **61**(9): p. 1205-1225.
9. Bayo, E., J.M. Cabrero, and B. Gil, *An effective component-based method to model semi-rigid connections for the global analysis of steel and composite structures*. Engineering Structures, 2006. **28**(1): p. 97-108.
10. Chen, C.-C., C.-C. Lin, and C.-L. Tsai, *Evaluation of reinforced connections between steel beams and box columns*. Engineering Structures, 2004. **26**(13): p. 1889-1904.
11. de Lima, L.R.O., et al., *Experimental and mechanical model for predicting the behaviour of minor axis beam-to-column semi-rigid joints*. International Journal of Mechanical Sciences, 2002. **44**(6): p. 1047-1065.
12. Schneider, S.P. and Y.M. Alostaz, *Experimental Behavior of Connections to Concrete-filled Steel Tubes*. Journal of Constructional Steel Research, 1998. **45**(3): p. 321-352.
13. Wald, F., et al., *Experimental behaviour of a steel structure under natural fire*. Fire Safety Journal. **In Press, Corrected Proof**.
14. Kabche, J.-P., et al., *Experimental characterization of hybrid composite-to-metal bolted joints under flexural loading*. Composites Part B: Engineering, 2007. **38**(1): p. 66-78.
15. Fu, F. and D. Lam, *Experimental study on semi-rigid composite joints with steel beams and precast hollowcore slabs*. Journal of Constructional Steel Research, 2006. **62**(8): p. 771-782.
16. Casafont, M., et al., *Experimental testing of joints for seismic design of lightweight structures. Part 2: Bolted joints in straps*. Thin-Walled Structures, 2006. **44**(6): p. 677-691.

17. Vigh, L.G. and L. Dunai, *Finite element modelling and analysis of bolted joints of 3D tubular structures*. Computers & Structures, Computational Structures Technology, 2004. **82**(23-26): p. 2173-2187.
18. Nethercot, D.A., *Frame structures: global performance, static and stability behaviour: General Report*. Journal of Constructional Steel Research, 2000. **55**(1-3): p. 109-124.
19. Chen, S. and G. Du, *Influence of initial imperfection on the behaviour of extended bolted end-plate connections for portal frames*. Journal of Constructional Steel Research. **In Press, Corrected Proof**.
20. Shakourzadeh, H., Y.Q. Guo, and J.L. Batoz, *Modeling of connections in the analyses of thin-walled space frames*. Computers & Structures, 1999. **71**(4): p. 423-433.
21. Shanmugam, N.E., et al., *Modelling of steel-concrete composite joints*. Journal of Constructional Steel Research, Second World Conference on Steel in Construction, 1998. **46**(1-3): p. 247-249.
22. Langdon, G.S. and G.K. Schleyer, *Modelling the response of semi-rigid supports under combined loading*. Engineering Structures, 2004. **26**(4): p. 511-517.
23. Rodrigues, F.C., A.C. Saldanha, and M.S. Pfeil, *Non-linear analysis of steel plane frames with semirigid connections*. Journal of Constructional Steel Research, Second World Conference on Steel in Construction, 1998. **46**(1-3): p. 94-97.
24. Wan-Shin, P. and Y. Hyun-Do, *Panel shear strength of steel coupling beam-wall connections in a hybrid wall system*. Journal of Constructional Steel Research, 2006. **62**(10): p. 1026-1038.
25. Neves, L.A.C., P.J.S. Cruz, and A.A.R. Henriques, *Reliability analysis of steel connection components based on FEM*. Engineering Failure Analysis, 2001. **8**(1): p. 29-48.
26. Kemp, A.R. and D.A. Nethercot, *Required and available rotations in continuous composite beams with semi-rigid connections*. Journal of Constructional Steel Research, 2001. **57**(4): p. 375-400.
27. Olsen, P.C., *Rigid-plastic finite element analysis of steel plates, structural girders and connections*. Computer Methods in Applied Mechanics and Engineering, 2001. **191**(8-10): p. 761-781.
28. Cheng, C.-T. and C.-C. Chen, *Seismic behavior of steel beam and reinforced concrete column connections*. Journal of Constructional Steel Research, 2005. **61**(5): p. 587-606.
29. Calado, L. and A. Lamas, *Seismic modelling and behaviour of steel beam-to-column connections*. Journal of Constructional Steel Research, Second World Conference on Steel in Construction, 1998. **46**(1-3): p. 258-259.
30. Turvey, G.J. and R.J. Brooks, *Semi-rigid--simply supported shear deformable pultruded GFRP beams subjected to end moment loading: comparison of measured and predicted deflections*. Composite Structures, 2002. **57**(1-4): p. 263-277.
31. Gizejowski, M.A., J.P. Papangelis, and H.C. Parameswar, *Stability design of semi-continuous steel frame structures*. Journal of Constructional Steel Research, Second World Conference on Steel in Construction, 1998. **46**(1-3): p. 99-101.
32. Zaharia, R. and D. Dubina, *Stiffness of joints in bolted connected cold-formed steel trusses*. Journal of Constructional Steel Research, 2006. **62**(3): p. 240-249.

33. Lim, J.B.P. and D.A. Nethercot, *Stiffness prediction for bolted moment-connections between cold-formed steel members*. Journal of Constructional Steel Research, 2004. **60**(1): p. 85-107.
34. Kudzys, A., *The structural safety of connections with fillet welds*. Journal of Constructional Steel Research. **In Press, Corrected Proof**.
35. Liew, J.Y.R., et al., *Testing of steel-concrete composite connections and appraisal of results*. Journal of Constructional Steel Research, 2000. **56**(2): p. 117-150.
36. Nawy, E.G., *Reinforced concrete : a fundamental approach*. 5th ed. 2003, Upper Saddle River, N.J.: Prentice Hall. xvii, 821.
37. MacGregor, J.G., *Reinforced concrete : mechanics and design*. 3rd ed. 1997, Upper Saddle River, N.J.: Prentice Hall. xiii, 939.
38. Mahendran, M. and C. Moor, *Three-Dimensional Modeling of Steel Portal Frame Buildings*. Journal of Structural Engineering, 1999. **125**(8): p. 870-878.
39. Kicinger, R., T. Arciszewski, and K. DeJong, *Evolutionary Design of Steel Structures in Tall Buildings*. Journal of Computing in Civil Engineering, 2005. **19**(3): p. 223-238.
40. Grierson, D.E. and S. Khajehpour, *Method for Conceptual Design Applied to Office Buildings*. Journal of Computing in Civil Engineering, 2002. **16**(2): p. 83-103.
41. Sarma, K.C. and H. Adeli, *Comparative Study of Optimum Designs of Steel High Rise Building Structures Using Allowable Stress Design and Load and Resistance Factor Design Codes*. Practice Periodical on Structural Design and Construction, 2005. **10**(1): p. 12-17.
42. Wu, J.-C., W.-C. Lu, and W.-C. Hsu, *Implementation of a Feasible Control Design Process Incorporating Robustness Criteria for Wind-Excited High-Rise Buildings*. Journal of Structural Engineering, 2006. **132**(1): p. 89-101.

INTEGRATION OF GPS MEASUREMENTS AND SECONDARY STRUCTURAL ELEMENTS INTO THE SERVICEABILITY DESIGN OF STRUCTURES

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Bing LI⁵, and Priyan A. MENDIS⁶

ABSTRACT: Preliminary studies have been undertaken to investigate if recent developments in GPS measuring performance now make this technology suitable for structural monitoring. A précis of this work along with a preliminary analysis of the application of such information into the design of high rise buildings is reported. In addition to reporting these preliminary studies, this paper outlines a hypothesis for the development of a modified design philosophy for high rise buildings that overcomes premature failure of non-structural components, under serviceability conditions, by aligning and thus optimising the design for non-structural and structural components. The paper outlines the background of current integrated design developments and considers a new design technique to predict and utilize the structural action of the total building system, not just the skeletal structure. The paper concludes with an outline of current and proposed research to confirm the potential whole of life cost savings.

KEYWORDS: G.P.S., high-rise buildings, design, non-structural components

1. INTRODUCTION

GPS is a measurement technology that finds application in a wide variety of civil and structural engineering projects. One particularly challenging application is the measurement of long term structural deformation and higher frequency modal behaviour of multi-storey buildings. The capabilities of GPS in this regard need further development to achieve appropriate levels of dependability and to provide a comprehensive picture of structural behaviour. However if existing limitations can be overcome, the potential of GPS structural monitoring to inform the engineering design process is substantial. This is a fundamental premise of the new high rise building design philosophy presented in the paper.

The detailed design of high-rise buildings (typically more than 40 storeys) is usually governed by serviceability limit state considerations rather than ultimate limit state factors. This contrasts with the current design practice in which the ultimate limit state along with the allowable serviceability limits is used to design the skeletal structure of a high-rise building. The predicted behaviour from this design dictates the required serviceability response of secondary elements. Serviceability limit states are normally dominated by wind effects and are checked for moderate earthquakes and torsional effects. The design of non-structural components (eg. partitions, blockwork, ceilings and mechanical

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services) is based on the assumption that they are isolated from the skeleton. However, construction practicalities and building functional requirements (eg. motion perception, vibration and noise attenuation) result in the secondary components being partially attached to the skeletal structure rather than being separate as assumed in the design.

2. DESIGN PHILOSOPHY

A high-rise building is defined as a multi-storey building which is tall enough to be affected by lateral forces due to wind, earthquakes or blasts to an extent that they play an important role in the structural design [1]. The design process of typical 30 to 50 story buildings involves designing a skeleton to resist the ultimate limit state loads and the serviceability limit state loads, including allowances for extreme wind and earthquakes. Structural engineers design high-rise buildings taking no account of (notionally) non-structural elements such as partitions; blockwork; doors; windows; ceilings; and mechanical services in the design process. However, buildings are widely recognized as a complex assemblage of both structural skeleton and non-structural components [2]. Non-structural components are considered by designers as infill or providing internal services based on the assumption that they are isolated from the skeleton.

The actual behaviour of high-rise buildings is very complex because of the conflicting requirements of diverse (structural and non-structural) building systems [3]. Thus, the traditional design approach for high-rise buildings is not accurate enough to predict actual performance.

In almost all high-rise buildings, the so-called “non-structural components” provide support in resisting lateral loads. Moreover, the interaction between non-structural and structure elements will significantly influence the overall performance of a high rise structure. Three dimensional analysis methods can help better understand the behaviour of high-rise buildings compared with traditional two-dimensional analyse. In various case studies [3], several well known high rise buildings from all over the world have been used to demonstrate the benefits of, not only integration between structural and architectural design, but also the integration of structural and non-structural components during the design process. These cases illustrate the importance of the role played by non-structural components in the overall performance of high-rise structures.

It is noteworthy that, during the development of design concepts, more and more designers have noted that the obvious interaction between structural and non-structural elements may have significant influence on structural performance.

The actual performance of real buildings differs significantly from that of idealised structural models [4-5]. Gad et al [6-8] have clearly shown that non-structural components in low-rise buildings can increase lateral stiffness and strength by more than 100%. Su et al. [2] found the contribution of non-structural elements to the overall stiffness of tall buildings in their case study could be reach to as much as 87%. These studies account for the difference between the theoretical estimates and real performance.

Several computer-based design optimization methods were widely discussed in order to optimize the design process so that cost-effective design methods and better structure performance could be achieved [9-12]. Although these proposed methods considered the integration of the structural skeleton and some of the non-structural elements they were still not able to accurately predict the actual behaviour of high rise structures under lateral loads.

In order to better understand the role played by non-structural components in influencing structural performance, it is necessary to measure and analyse the actual performance of high-rise buildings against the predicted performance from structural models. However the process of obtaining accurate, reliable and comprehensive measurement data to give a complete picture of structural performance is a

complex and demanding one. In fact this requirement falls at the leading edge of high precision engineering surveying and is the subject of research at a number of institutions. Researchers at the University of New South Wales, The University of Nottingham and the University of Calgary are working on the structural performance of bridges using such advanced techniques. However, no such research has been reported on high rise buildings.

3. PROPOSED NEW DESIGN TECHNIQUE

The lateral peak acceleration limits and inter-storey drift are normally used as the main serviceability criteria in the design of high rise buildings. However, facade systems are renowned for being drift intolerant [13]. Moreover, there is limited understanding of the actual acceleration experienced in buildings due to a serious lack of full scale data on building response. The proposed new design technique seeks to overcome this deficiency by integrating state-of-the-art GPS measurement technology with real time dynamic measurement of buildings.

Some of the limitations of early GPS measurement technology have now been overcome thus extending the capture of data from external visual targets to include shadowed and internal zones of buildings, thus facilitating this new design approach. The structural performance data logged by this type of measurement system enables the actual stiffness and load paths within high-rise buildings to be determined. The ultimate load carrying capacity of a building which is based on the skeletal structure remains unchanged.

The proposed approach is innovative in that it will:

- integrate real time radio based, GPS and accelerometer measuring systems;
- incorporate an integrated design approach involving structural and non-structural components;
- optimise building design potentially leading to safer buildings constructed in a more cost effective way;
- provide improved understanding of actual building displacements and enhance the design and integration of secondary components into the overall building system;

A preliminary investigation has identified potential for the design concept called *evolutionary* design to cover integrated design for the skeletal structure and non-structural components of high-rise buildings. Details of recent developments in GPS and an example of its application follow.

4. RECENT DEVELOPMENT IN GPS

The variety of applications for which GPS is being employed continues to expand. As the technology becomes more affordable and simple, the use of GPS for recreational applications is growing rapidly. At the same time, the versatility and potential accuracy of GPS gives rise to new professional and scientific applications on a routine basis. In the field of civil and structural engineering, GPS has been used in a variety of ways and for a diverse range of tasks in the design, construction and post-construction phases of major projects. Of particular interest, in the context of this paper, is the use of GPS for detecting structural deformation and vibration of multi-storey buildings [14-15]. This is a particularly challenging application for two reasons. First, the accuracy requirements can be very demanding. Second, and of more concern, the measurement environment in a structural setting is often not conducive to the acquisition of high quality measurement data. Obstructions reduce the number of visible satellites and reflective surfaces in the vicinity of the GPS antenna cause signal interference. These factors combine not only to limit when and how GPS can be used but also to hinder the accuracy that can be achieved.

Optimising the use of GPS for structural deformation monitoring has been and continues to be the focus of much research [16-17]. A common objective is to investigate ways of augmenting GPS with

a complementary measurement technology. To this end, many researchers have employed accelerometers in tandem with GPS to create a more comprehensive picture of structural behaviour [18]. A benefit of accelerometers is that they are well suited to detecting high frequency structural motion and also facilitate the measurement of structural behaviour in areas where it is difficult if not impossible to collect GPS data. GPS on the other hand provides a very reliable and robust way to remotely and continuously monitor absolute structural displacements.

Advances in GPS technology, in particular the advent of high-rate GPS receivers capable of collecting observations at 50-100 Hz, mean that GPS now has the potential to detect the high frequency modal behaviour of an engineering structure. Research is also progressing in the area of multipath detection and mitigation [19]. Strategies to deal with signal obstructions are also being investigated, this being a particular focus of research being conducted at the University of Melbourne.

5. PRELIMINARY APPLICATION OF GPS

Recent research into GPS for structural deformation monitoring has investigated the capabilities of reasonably high rate (10 Hz) GPS receivers to detect high frequency structural motion of Melbourne's Westgate Bridge. Details of this study and discussion of initial results are presented in Raziq and Collier [20].

The Westgate Bridge is a cable-stayed box girder bridge constructed across the Yarra River in the mid 1970's. As well as being a prominent landmark, the bridge provides a key vehicular link between the western suburbs and the city's central business district. Estimated traffic volume is currently about 160,000 vehicles per day [21]. The Westgate Bridge consists of five steel spans with concrete approaches making a total length of 2590 m. As shown in Figure 1, the central steel spans are supported by a combination of cables and concrete piers. The cables are suspended from two steel towers, each rising to a height of 45.75 m above the deck of the bridge. The bridge deck is 58.61 m above the Yarra River and has a width of about 37.34 m.

The project compared GPS data collected at the three stations shown in Figure 1 to accelerometer data from nearby sites and an earlier wind tunnel analysis of a model of the bridge [22].



Figure 1 – View of the central steel spans of the Westgate Bridge, showing the approximate locations of three GPS receivers used in the monitoring study.

The objective of the study was to assess the performance of GPS in identifying dominant modal frequencies of the structure. As briefly discussed below, the initial results are very promising. For the purposes of the analysis, the original 10 Hz GPS data was re-sampled at 2 Hz since power spectral density analyses using 10 Hz, 5 Hz and 2 Hz data revealed that the 2 Hz data yielded dominant frequencies with maximum power. Compared to the accelerometer data, the GPS data was very noisy, presumably due to the impact of multipath, particularly as expected at Stations 1 and 2. Notwithstanding the high levels of noise, power spectral density analysis successfully identified the modal frequencies shown in Table 1.



Figure 2a – GPS receiver at Station 1



Figure 2b – GPS receiver at Station 3

These preliminary results from the Westgate Bridge study support the premise that GPS can be successfully employed to monitor the structural behaviour of multi-storey buildings. Not only is GPS capable of determining long term displacements, higher frequency modal behaviour is also detectable. In addition to integrating GPS and accelerometer data, future research will focus on extracting reliable high frequency data from new generation GPS receivers, mitigating the influence of repeated multipath effects and combining multiple GPS receivers to account for dominant signal obstructions. The results from the proposed GPS research will assist engineers in understanding the impact of non-structural components on building behaviour and will thus contribute to the development of a refined design procedure that takes these components into account.

6. PROPOSED MODELLING

Similar to the case study presented in Section 5, in this research a high rise building will be instrumented using both GPS and accelerometers to measure the building response over a period of time. These measurements will reveal the true dynamic characteristics of the building including natural frequencies, mode shapes and damping. These properties will be used to validate a non-linear Finite Element (FE) model of the same building. The FE model will incorporate the main non-structural components such as partition walls and façade panels. These components will be modelled as non-linear springs which will be connected to the skeletal structure as if they are diagonal bracing elements. The load-deflection characteristics will be obtained from previously completed research [6-7 & 23-24]. Indeed, similar modelling technique has been successfully adopted for low rise residential structures [8]. As part of the modelling work, an extensive parametric study will be conducted to examine possible interaction scenarios between the skeletal structure and non-structural components including a range of values for the stiffness, strength and degradation of non structural components.

Table 1 – Results summary from the Westgate Bridge GPS & accelerometer trials

Definition	Frequency	GPS	Accelerometers	Model
At Station 1 (Bridge Deck @ ½ Span) – Vertical displacements only				
1 st vertical bending	0.35 Hz	✓	✓	✓
3 rd vertical bending	1.02 Hz	✗ ¹	✓	✓
Undefined	0.79 Hz	✗ ¹	✓	✗
At Station 2 (Bridge Deck @ ¼ Span) – Vertical displacements only				
1 st vertical bending	0.35 Hz	✓	✓	✓
2 nd vertical bending	0.53 Hz	✗ ¹	✓	✗ ²
3 rd vertical bending	1.02 Hz	✗ ¹	✓	✓
Undefined	0.79 Hz	✗ ¹	✓	✗
At Station 3 (East Tower) – Longitudinal displacements only				
Undefined	0.12 Hz	✓	N/A ³	N/A ⁴
1 st vertical bending (deck) ⁵	0.34 Hz	✓	N/A ³	N/A ⁴
Undefined	0.51 Hz	✓	N/A ³	N/A ⁴
At Station 3 (East tower) – Lateral displacements only				
Undefined	0.51 Hz	✓	N/A ³	N/A ⁴
Undefined	0.55 Hz	✓	N/A ³	N/A ⁴
Undefined	0.62 Hz	✓	N/A ³	N/A ⁴

Table notes:

1. GPS was not able to identify the higher frequency vertical bending moments predicted from the wind tunnel analysis and confirmed from the accelerometer data because of the high noise levels. Further investigation using higher rate GPS data may overcome this limitation, as may subsequent research to remove some of the repeated multipath effects.
2. The 2nd vertical bending of the deck was not observed in the wind tunnel analysis of the bridge, but it was predicted. This study confirmed its existence based on the accelerometer data.
3. No accelerometer data could be collected in parallel with the collection of GPS data at the top of the East tower due to equipment failure.
4. The wind tunnel analysis of the bridge did not consider the modal frequencies of the bridge towers.
5. In the longitudinal direction, the tower demonstrated a modal frequency matching the 1st vertical bending frequency of the bridge deck. This is an expected finding and further validates the GPS results.

7. CONCLUDED REMARKS

This paper has established that there is opportunity to design high-rise buildings using an integrated approach incorporating all components of the building. Such an approach has the potential to ensure that the skeleton of the building fulfils the ultimate limit state design requirements and that the non-structural components contribution to the serviceability limit states is recognised. This will result in overall savings in the skeletal structure and improved design of the non-structural components.

The impediment to achieving such integrated design has been the measurement of actual building performance and establishing the contribution of non-structural components to the building stiffness. The preliminary application of G.P.S. measuring technology to overcome this deficiency is most encouraging.

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REFERENCES

1. Stafford Smith, B. and Coull, A., *Tall building structure: analysis and design*, Wiley, New York, 1991.
2. Su, R. K. L., Chandler, A. M., Sheikh, M. N. and Lam, N. T. K., "Influence of non-structural components on lateral stiffness of tall buildings", *The Structural Design of Tall and Special Buildings*, Vol. 14, 2005, pp. 143-164.
3. Sev, A., "Integrating Architecture and Structural Form in Tall Steel Building Design", *CTBUH REVIEW*, 2001, Vol. 1, No. 2.
4. Naeim F., "Lessons learned from performance of non-structural components during the January 17, 1994 Northridge Earthquake – case studies of six instrumented multistory buildings", *Journal of Seismology and Earthquake Engineering*, Vol. 2(1), 1999, pp 47-57
5. Sugiyama T., Uemura, M., Fukutama, H., Nakano, K. and Matsuzaki, Y., "Experimental study on the performance of the RC frame infilled cast-in-place non-structural RC walls retrofitted by using carbon fibre sheets", Proceedings of the 12th WCEE, Auckland, New Zealand, 2000, (Paper No 2153).
6. Gad, E.F., Chandler, A.M, Duffield, C.F. and Stark, G., "Testing of cold-formed steel framed domestic structures", Proceedings of the 11th European Conference on Earthquake Engineering, France, 1998.
7. Gad, E.F., Duffield, C.F., Hutchinson, G.L., Mansell, D.S., and Stark, G., "Lateral performance of cold-formed steel-framed domestic structures", *Engineering Structures*, Vol. 21(1), 1999, pp 83-95.
8. Gad, E.F., Chandler, A.M., Duffield, C.F., Stark, G., "Lateral Behaviour of Plasterboard-Clad Residential Steel Frames", *Journal of Structural Engineering (ASCE)*, Vol. 125(1), 1999, pp 32-39.
9. Kicinger, R., Arciszewski, T. and DeJong, K., "Evolutionary Design of Steel Structures in Tall Buildings", *Journal of Computing in Civil Engineering*, Vol. 19, 2005, pp. 223-238.
10. Grierson, D. E. and Khajehpour, S., "Method for Conceptual Design Applied to Office Buildings" *Journal of Computing in Civil Engineering*, Vol. 16, 2002, pp. 83-103.
11. Sarma, K. C. and Adeli, H., "Comparative Study of Optimum Designs of Steel High Rise Building Structures Using Allowable Stress Design and Load and Resistance Factor Design Codes", *Practice Periodical on Structural Design and Construction*, Vol. 10, 2005, pp. 12-17
12. Wu, J.-C., Lu, W.-C., and Hsu, W.-C., "Implementation of a Feasible Control Design Process Incorporating Robustness Criteria for Wind-Excited High-Rise Buildings", *Journal of Structural Engineering*, Vol. 132, 2006, pp. 89-101
13. McBean, P.C., "Drift intolerant façade systems and flexible shear walls. Do we have a problem", *Annual Technical Conference of the Australian Earthquake Engineering Society Albury, NSW*, 2005, pp35:1-7.
14. Celebi, M., "GPS in dynamic monitoring of long-period structures," *Soil Dynamics and Earthquake Engineering*, Vol. 20, 2000, pp. 477-48.
15. Brownjohn, J.M.W., C. Rizos, G.H. Tan, and T.C. Pan, "Real-time Long Term Monitoring of Static and Dynamic Displacements of an Office Tower, Combining RTK GPS and Accelerometer Data". *Presented at 1st FIG International Symposium of Engineering Surveys for Construction Works and Structural Engineering*, Nottingham, United Kingdom, 2004.

16. Leach M.P. and M. Hyzak, "GPS structural monitoring as applied to cable-stayed suspension bridge". *Presented at XX FIG Congress*, Melbourne, Australia, 1994.
17. Larocca, A.P.C., "Using high-rate GPS data to monitor the dynamic behavior of a cable-stayed bridge". *Presented at 17th International Technical Meeting of the Satellite Division of the U.S. Institute of Navigation*, ION GPS / GNSS 2004, Long Beach, California, USA, 2004.
18. Li, X., G.D. Peng, C. Rizos, L. Ge, Y. Tamura, and A. Yoshida, "Integration of GPS, accelerometers and optical fibre sensors for structural deformation monitoring", *presented at International Symposium on GPS/GNSS*, Tokyo, Japan, 2003.
19. Roberts, G.W., X. Meng, and A.H. Dodson, "Using Adaptive Filtering to Detect Multipath and Cycle Slips in GPS/Accelerometer Bridge Deflection Monitoring Data", *presented at FIG XXII International Congress*, Washington, D.C., USA, 2002.
20. Raziq, N and Collier, P.A., "GPS Deflection Monitoring of the Wesgate Bridge", *Add details of FIG/IAG Conference in Baden*, Austria, May 22-24, 2006.
21. Baker, R., "West Gate review's revealing findings", *The Age*, 2005, Melbourne.
22. Balfe, P.J., A.B. Jackson, W.H. Melbourne, A.P. Sewell, A.R. Toakley, D.G. Williams, and H. G.Wolfram, "Redesign of West Gate Bridge, Melbourne", Road Construction Authority Victoria Australia, 1986.
23. Griffith, M.C., and Alaia, R., (1997) "Gap Effects on the Seismic Ductility of Brick Infilled Concrete Frames", *Proceedings of the 15th Australasian Conference on the Mechanics of Structures and Materials*, Melbourne, pp 305 - 310.
24. Liew, Y.L., Gad, E.F., and Duffield C.F., (2002) "The influence of plasterboard clad walls on the structural behaviour of low rise residential buildings", *Electronic Journal of Structural Engineering* Vol. 1, pp 1-16.

***APPENDIX II:
DESIGN OF BOLTS, PIN CONNECTIONS AND
WELDS FROM AS 4100 (SECTION 9)***

		Bolt	Pin	Weld
Strength	Shear	$f_{vf} = 0.62 f_{uf}$ $V_{fn} = 0.62 A_c f_{uf}$ $V_{fs} = 0.62 A_0 f_{uf}$ <i>For bolted lap splice connections, a reduction factor k_r can be used</i>		Fillet welds $\sqrt{v_n^{*2} + k_v (v_v^{*2} + v_{vt}^*)} = k_w (0.6 f_{uw} t_t)$ $= \phi k_w v_w$
		$k_r = 1.0$ $l_j \leq 15d_f$ $= 1.075 - \frac{l_j}{200 d_f}$ $15 d_f < l_j \leq 65d_f$ $= 0.75$ $l_j > 65d_f$		$k_v = 1.0$ $k_w = 1.0$ An alternative approach is to use a load-deformation method. Plug and slot welds Be used to transmit shear in a lap joint or to prevent the buckling of separation of the plates in a lap joint. The use is not extensive for structural applications. <i>Diameter of the hole for a plug weld:</i> no less than the thickness of the part containing it plus 8mm. The diameter should not exceed either the minimum diameter plus 3mm or 2.25 times the thickness of the part, whichever is greater <i>The minimum center to center spacing of plug welds :</i> 4 times the diameter of the hole <i>Depth of the filling of plug welds:</i> in material 16mm or less should be equal to the thickness of the material. For over 16mm, should be at least one half of the thickness of material, but no less than 16mm. <i>Length of the slot:</i> should not exceed 10 times the thickness of the part containing it. <i>Width of the slot:</i> should not exceed either the minimum width plus 3mm, or 2.25 times the thickness of the part, whichever is the greatest. <i>The ends of the slot:</i> semicircular or have corners round to a radius not less than the thickness of the part containing it. Except those ends extend to the edge of the part. <i>The minimum spacing of lines of slot welds</i> in a direction transverse to their length should be 4 times the width of the slot. <i>The minimum center to center spacing</i> in a longitudinal direction on any line should be 2 times the length of the slot.
			The nominal shear capacity is based on a shear stress at failure of 62% of the yield stress of the pin material, as for a bolt subject to shear force	

Tension

$$N_{tf} = A_s f_{uf}$$

Combination

- Elliptical interaction relationship
- The nominal tension capacity and the nominal shear capacity used in the denominators of the interaction equation are the respective nominal capacities of the bolt under the separate individual loads, with the nominal shear capacity being dependent upon the locations of the shear planes, s for a bolt subject to shear force alone.

Bearing

Is not considered as a possible failure mode

The relatively low failure stress of 1.4 times the yield stress of the pin material reflects the critical nature of this load on a single pin. The factor k_p of 0.5 for a pin that allows rotation reflects the fact that continual movement of the pin plates around the pin circumference creates a wearing effect.

A pin is treated as a compact member, subject only to plastic yielding.

Ply in bearing

Long end distance in the direction of applied loads

$$V_b = 3.2 f_{up} d_f t_p$$

Use definition as bolt

Short end distance: plate-tear out failure

$$V_b = a_e t_p f_{up}$$

Elastic analysis:

$$V_n^* = \frac{r_n}{r_{\max}} V_{of}^*$$

$$x_e = \frac{-\left(\sum y_n^2 + \sum x_n^2\right)}{n_b e}$$

$$y_e = 0$$

$$V_{of}^* = \frac{-V^* r_{\max}}{\sum (x_n - x_e)}$$

$$V_{bn}^* = \frac{V^*}{n_b}$$

$$V_{bc}^* = \frac{(V^* e)_n}{\left(\sum x_n^2 + \sum y_n^2\right)}$$

Group assessment

Plastic analysis

Assume all bolts not at the centre of rotation are deformed sufficiently to become fully plastic and all transmit the same force and failure. Other methods

- In-plane loading:
 - Linear elastic method
 - Alternative method
- Out-of-plane loading:
 - Methods identified in other documents

Serviceability

- *The maximum amount of slip on connections (not classified as slip-critical: 2-3 mm.*
- *Considerable variation in both the initial bolt tension N_i and the function of the surface condition of the interfaces u depend on the bolt grade and the method of installation.”*

Design details

- Minimum pitch: 2.5 bolt diameters
- Minimum edge distance: be controlled by end plate tear out
- Maximum pitch: based on successful past practice
- Maximum edge distance: based on successful past practice, also intended to prevent any potential curling up of plate edges



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