OPTIMISATION OF LATERAL LOAD-RESISTING SYSTEMS IN COMPOSITE HIGH-RISE BUILDINGS

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Abstract

The use of composite structures in buildings is time effective, cost efficient and provides column-free space. Composite construction technology is gaining popularity among builders, contractors and developers not only in Australia, but throughout the world. Academic research on this subject mainly focuses on either reinforced concrete or structural steel buildings. Even though studies of individual composite elements of structure (such as composite columns and composite beams) are in abundance, there is a scarcity of research related to the structural performance of composite buildings as a complete structure. The civil/structural engineer has to go through a lengthy process of modelling and detailed calculation to find out the requirements of belt-truss and outriggers and to establish locations of these in buildings. Hence, this topic needs to be investigated thoroughly at the academic level to be able to occupy an absolute position in standards and codes of practice.

This research was carried out by using Finite element modelling of building prototypes with three different layouts (rectangular, octagonal and L-shaped) for three different heights (98 m, 147 m and 199.5 m). Variations of lateral bracings (varied number of belt-truss and outrigger floors and varied placements of belt-truss and outrigger floors along model height) with RCC (reinforced cement concrete) core wall were used in composite high-rise building models. Models of composite buildings were then analysed for dynamic wind and seismic loads. The effects on serviceability (deflection, storey drift and frequency) of models were studied.

The best model options among analysed models were outlined with respect to belt-truss and outrigger placements and horizontal loadings. Analytical models were proposed using a maximum height model for prediction of deflection.

It was found out that provision of top level single floor belt-truss and outrigger would be very beneficial for buildings up to 150 m height, if subject to seismic load while; under wind loads, provision of belt-truss and outriggers at mid-height would provide better displacement control. Multi-storeys between 150 m to 200 m height respond well with single floor bracings placed at 2/3rd building height (measured from base). However; if a level of double floor lateral bracings was needed then bracings worked well at the top level of the building with critical earthquake
loadings. It was also observed that staggered levels of outriggers, i.e. two or three single truss floors at various heights such as mid-height and 2/3rd height (measured from base) of building rendered better lateral deflection control than double floor belt-truss and outriggers in buildings between 150 m to 200 m height.
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$A = \text{plan area (m}^2\text{).}$

$A_c = \text{area of concrete.}$

$A_g = \text{gross area of section.}$

$A_{ST} = \text{area of steel.}$

$A_{\text{wall}} = \text{cross-sectional area of shear wall (m}^2\text{).}$

$b = \text{breath of plan layout (m).}$

$C(T) = \text{elastic site hazard spectrum.}$

$C_{d(T)} = \text{horizontal design response spectrum as a function of (T).}$

$C_{\text{fig,e}} = \text{external component was selected for structures having “h > 25m”}.$

$C_{p,i} = \text{internal component selected for “All walls are equally permeable”.}$

$\text{CQC} = \text{Complete Quadratic Combination.}$

$d = \text{depth of plan layout (m).}$

$E = \text{elastic modulus (MPa).}$

$E = \text{site elevation above mean sea level.}$

$E_c = \text{elastic modulus of concrete.}$

$E_{s} = \text{elastic Modulus of steel.}$

$E_T = \text{elastic modulus of transformed section.}$

$f = \text{frequency (Hz).}$

$\text{FEA} = \text{Finite Element analysis.}$

$\text{FEM} = \text{Finite Element modelling.}$

$f_n = \text{natural/fundamental frequency.}$

$G = \text{gravitational loads.}$
\( G_i \) = permanent action i.e. self weight or dead load.

\( H \) = building height (m).

\( \text{H}_{\text{floor}} \) = floor to floor height (m).

\( h_i \) = of \( i \)th level above the base of structure in meters.

\( \text{HS} \) = horizontal design response spectrum.

\( k \) = stiffness (kN/m).

\( k \) = exponent dependent on the fundamental natural period of structure.

\( K_a \) = area reduction factor.

\( K_{c,e} \) = combination factor applied to external pressure.

\( K_{c,i} \) = combination factor applied to internal pressure.

\( k_{F,i} \) = seismic distribution factor for the \( i \)th level

\( K_l \) = local pressure factor.

\( K_p \) = porous cladding reduction factor.

\( k_p \) = probability factor appropriate for the limit state under consideration.

\( m \) = mass (kg).

\( M_h \) = hill shape multiplier.

\( M_{\text{lee}} \) = lee effect multiplier considered.

\( M_{z,\text{cat}} \) = height and terrain multiplier.

\( n \) = no. of levels in structure.

\( Q_i \) = imposed action (live load).

\( \text{RCC} \) = reinforced cement concrete.

\( S_p \) = structural performance factor.

\( \text{SRSS} \) = Square Root of Sum of Square.

\( \text{SS} \) = site-specific design response spectrum.
UB = Universal Beam.
UC = Universal column.
V = base shear (kN).
WB = Welded beam.
WC = Welded column.
$W_i$ = seismic weight at $i$th level (kN).
$W_j$ = seismic weight of structure or component at level $j$ (kN).
$W_t$ = total seismic weight of building (kN).
$W_x$ = wind in X-direction.
$W_y$ = wind in Y-direction.
X-dir. = X-direction.
Z = earthquake hazard factor.
$\beta$ = Strand7 seismic factor related to base shear.
$\mu$ = structural ductility factor.
$\pi = 3.1415926535$.
$\mu$ = structural ductility factor ($\mu = \mu_0$).
$\psi_c$ = imposed action (live load) combination factor.
$\Delta$ = deflection (mm).
$\delta$ = inter-storey drifts (mm).
$\gamma_c$ = density of concrete.
$\gamma_s$ = density of steel.
$\gamma_T$ = density of transformed section.
Statement of Original Authorship

The work contained in this thesis has not been previously submitted to meet requirements for an award at this or any other higher education institution. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made.

Signature: QUT Verified Signature
Date: 19/01/2014
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List of Publications


Chapter 1: Introduction

1.1 PROLOGUE

This thesis aims to study the effect and outcomes of horizontal force applied to composite multi-storey braced frame structures. The bracing is provided in the form of a concrete wall, structural steel belt-truss and outriggers.

The study has been carried out by using the latest computer modelling technology, Finite Element Modelling (FEM). FEA (finite element analysis) is already embedded in the engineering profession and academies of Australia and throughout the world.

1.2 PERSPECTIVE OF THESIS TOPIC

The main building materials are timber, masonry, steel and concrete. Timber and masonry have been stasis due to their limited capabilities, whereas concrete and steel have been transformed from Joseph Monier pots to sky-high buildings such as Burj Khalifa.

The beginning of composite construction is attributed to the year 1894, when concrete-encased beams were first used in a bridge in Iowa and a building in Pittsburgh, USA. Gradually this technology extended to Canada and Japan and then throughout the world. From the time it was first used, the composite system has been acknowledged as undeniably competent technique for enhancing structural performance. A large number of steel structures are now being designed compositely due to the efficiency of concrete shear wall in lateral load resistance. Chifley tower in Sydney, Australia and Jim Mao tower in Shanghai, China are two examples among many composite constructions.

1.3 GAPS IN ACADEMIC WORK

The use of composite structures in buildings is time effective, cost efficient and provides column-free space, and is highly suitable for commercial usage.
Although the guidelines for composite beam design are provided in Australian Standards (AS 2327.1, 2003), there is a scarcity of documents that explain the structure of the whole building.

There has been an abundance of academic work conducted on various composite elements, such as composite columns of various shape, and beams. Many tests have been conducted on circular, rectangular and square columns and the investigations by scientist and researcher continue. Yet there is a huge lag of scholarly items on the overall behaviour of buildings constructed using composite slab, beam and columns. Moreover, academic literature concentrates on the characteristics and properties of wind and earthquake loadings.

The structural designer has to go through a lengthy process of modelling a whole building prototype as there are no set procedures for finding out the requirements of outriggers and establishing the location of these in buildings. The procedure is usually based on trial and error as well as past experience. If a project is delayed or cancelled, the extensive work already performed to establish the feasibility of the project and the initial cost estimation can go to waste.

Therefore, this thesis aims to study the behaviour of composite buildings braced with shear walls and steel trusses under horizontal loads. This will not only be advantageous in the formulation of basic principals or rules for typical building structures within the scope of Australian standards, but will also help civil/structural engineers in their routine calculations of cost and material estimation at the conceptual/preliminary stage of the project.

1.4 AIM AND OBJECTIVES

To address the lack in research of composite building behaviour under horizontal loadings, the following objectives were established:

- To develop Finite Element modelling of building prototypes with three different layouts (rectangular, octagonal and L-shaped) and three heights (98 m, 147 m and 199.5 m) and validate the models with manual calculations.

- To perform a parametric study by varying the location of belt-truss and outriggers in high-rise composite building models.
To perform dynamic analyses of composite buildings subjected to wind and seismic loads.

To determine the best location of possible belt-truss and outriggers arrangement by comparison of results for wind and seismic action.

To develop an analytical model by using results from parametric study.

These objectives are undertaken through rigorous analysis of models in Strand7 (R2.4.4, 2011) in the subsequent chapters. The horizontal/lateral loadings are defined and then calculations are performed accordingly. The model verifications are carried out and finally results are extracted and compared and conclusions are drawn.

1.5 THESIS OUTLINE

The thesis consists of six chapters, inclusive of chapter 1. It provides a detailed review, description, calculation and analysis of the selected topic through the chapters outlined below.

Chapter 1 sets aims and objectives of thesis. It gives introduction of work performed in succeeding chapters to achieve targets of this study.

Chapter 2 provides a detailed review of construction in the context of composite buildings and their historical and modern background. A review of available literature for composite construction is conducted. The bracing system popular in composite construction is described. The provision of concrete core wall coupled with outriggers and belt-truss is scrutinized in detail with respect to the thesis modelling. The chapter also gives an account of research on the lateral/horizontal loads applied to buildings. The loads that mainly affect multi-storey constructions are wind and seismic loads.

Chapter 3 describes the setup of models. It includes calculations of transformed properties of composite elements. The range of layouts and prototypes, adopted variations of belt-truss and outriggers, disparity of storey heights and different layouts are described. The selected programme (Strand7 R2.4.4, 2011) and method of computer analysis are explained. Gravity loads for multi-storey buildings are also discussed in this chapter.

Chapter 4 covers wind load and choice and justification of load type (i.e. static or dynamic). The variables and their rationalisation are selected for analysis, and the
calculations specific to prototypes and load application in the programme (Strand7 R2.4.4, 2011) are summarised. The results are then extracted and given in tabulated format, and also represented graphically; conclusions from the analysis are presented.

Chapter 5 centres on the topic of seismic load calculation and its application in the software. An excerpt of various parameters and variables for earthquake actions is provided, as well as reasons for the selection of these parameters. Comprehensive calculations of seismic load within the scope and limitation of Australian Standard (AS 1170.4, 2007) is given. The results are listed in tables and graphs. Conclusions are presented at the end of the chapter.

Chapter 6 provides the results and conclusions drawn from the research. The outcomes of the thesis are provided in the form of the best options for models of composite buildings under lateral loadings. Moreover, formulae for predicting deflection are proposed as a product of the rigorous analysis conducted in the thesis. Suggestions for future research are recommended and discussed.
Chapter 2: Literature Review

2.1 INTRODUCTION

The concept of tall structures is not new to the world, yet the trend of high-rise construction started in the nineteenth century. High-rise or multi-storey buildings are being constructed either to cater for a growing population or as a landmark to boost a country’s name and get recognition.

The choice of thesis topic is examined and argued in the context of the research background, with examples of real-life structures. A description of composite constructions is given. This chapter emphasises that the scholarly material available usually deals with individual components of composite buildings, such as composite columns or composite beams. Moreover, the academic literature concentrates on the characteristics and properties of wind and earthquake loadings. There is little academic work on the overall behaviour of composite buildings under horizontal loadings.

2.2 MULTI-STOREY CONSTRUCTION

The onset of modern buildings can be traced back to the nineteenth century. High-rise buildings have become characteristic of commercial districts or cities. These are the result of meticulous thinking and precise design to accommodate a large number of people and supply all the modern day amenities to the occupants.

Ali (2001) pointed out that tall buildings emerged in late nineteenth century in the United States of America. Today, however, they are a worldwide architectural phenomenon, especially in Asian countries, such as China, Korea, Japan, United Arab Emirates, Singapore and Malaysia.

Mendis & Ngo (2008) proposed that this demand is always auxiliary to a multitude of variables, such as strength, durability, forming techniques, material characteristics, nature, aesthetics and much more. However, the design intent has always been to accomplish structures deemed to be affordable and safe during their life span.
Any structure, to be reliable and durable, must be designed to withstand gravity, wind, earthquakes, equipment and snow loads, to be able to resist high or low temperatures, and to assimilate vibrations and absorb noises.

Gabor (2006) stated that the main aim of structural design is to provide a safe load path during any stage of construction, as well as for the building’s life-span and during its demolition, under all possible loads and effects and within acceptable risk limits as set by the society.

According to Khan (1972) the performance of any structure depends upon following:

- Lateral sway criteria;
- Thermal movements;
- Structural and architectural interaction.

The main and primary concern is the stability and reliability of the entire structure and structural components, as well as their ability to carry applied loads and forces. Tall and lean buildings are more susceptible to lateral sway and deflections. The minimum limit to structural sizes suggested by various codes and standards are usually enough to support the weight of the building as well as the imposed dead loads and live loads. However, the real challenge for the structural engineer is to find out the structural behaviour of a building under wind and seismic actions. The effects of these external horizontal forces are highly unpredictable, and these mainly depend on building shape, size, mass, floor plan layout, and climatic conditions.

2.3 RESEARCH PROBLEM

This research aims to study the behaviour of composite multi-storey buildings under horizontal loads using belt-truss and outriggers as secondary bracings. This topic is analysed in the context of available academic material and the gaps in academic research are pointed out. This research is targeted to fill in the deficiency of scholarly material with respect to the thesis topic.

2.3.1 Previous work on composite building elements

A wide range of scholarly documents and numerous research works are available to investigate stress, failure mechanisms, durability and strength etc. of the
independent components of composite structural systems such as slabs, beams and columns. For instance;.

Liang et al. (2005) studied the strength of concrete filled steel box columns with a variety of square and rectangular shapes, using the fibre element analysis (where the composite section is discretized into many small regions called fibres).

Sandun et al. (2009) explored the impact of dynamic loadings on composite floors through finite element modelling in ABAQAS.

Ellobody et al. (2011) studied eccentrically loaded composite columns. Concrete filled steel tubes were used in this research. The authors have checked the strength of the columns under varied conditions of eccentricity and compared results with Euro Code 4.

The performance of composite columns under high temperature was studied by Young et al. (2011). The authors have utilised a non-linear three-dimensional finite element model for research using Euro Code 4. They have used a universal column (UC) section in a reinforced concrete square column.

Academic research has limited amount of material on overall performance of composite buildings, however; appreciable amount of literature is present on reinforced concrete and steel structures, such as;

Kian et al (2001) extrapolated the efficiency of belt-truss and outriggers in concrete high-rise buildings subjected to wind and earthquake loadings. Authors used two dimensional 40-storey model for wind and three dimensional 60-storey model for seismic load analysis. They came up with the optimum location of belt-truss and outriggers with 65% and 18% lateral deflection reduction for wind and earthquake loadings respectively.

Hoenderkamp et al (2003) presented a graphical method of analysis of tall buildings frames braced with outriggers and subjected to uniform lateral loadings. Authors have used steel structures for their two dimensional model. They have concluded that behaviour of steel braced frame with outriggers was similar to concrete wall with outriggers beams and further suggested that horizontal deflection and bending moments were influenced by stiffness and therefore; it should be included in the preliminary design of tall structures.
Hoenderkamp (2007) derived an analytical method for preliminary design of outrigger braced high-rise shear walls subjected to horizontal loading. He used a two dimensional analytical model of shear wall with outriggers at two levels, one outrigger has a fixed location up the height of the structure, while the second was placed at various location along the model height. He has given comparison of deflection reduction for a 29-storey model with few combination of two outriggers floors and concluded that the optimum location of the second outrigger was at $x/H = 0.577$ when the first one was placed at the top, i.e. $a/H = 0.0$.

Lee et al (2008) focused on deriving the equations for wall-frame structures with outriggers under lateral loads in which the whole structure was idealized as a shear-flexural cantilever and effects of shear deformation of the shear wall and flexural deformation of the frame were considered. Authors have verified the equation by considering the concrete wall-frame building structure under uniform wind loading. Conclusions highlighted that consideration of shear deformations of walls and flexural deformations of frame in analytical formula gave sufficiently accurate results.

Rahgozar (2009) presented mathematical model for calculation of stresses in columns of combined framed tube, shear core and belt-truss system. He applied his mathematical models to 30, 40 and 50 storey buildings and compared the results with SAP 2000 software for its applicability. He concluded with the best outrigger location at 1/4th and 1/6th of model height. His study was based on pure numerical models and he did not use the actual properties of materials i.e. concrete or steel or composite. He also did not use a realistic building layout but based his finding on assumptions of certain properties.

All the above researches do not consider a comprehensive study of composite structural system of dissimilar plan layouts of varied heights with different combinations of belt-truss and outriggers. Different combination of lateral load resisting system i.e. single floor or double floor bracings, with varied plan layouts and assorted heights models would results differently.

2.3.2 Review of academic work pertaining to wind actions

The history of tall buildings whether in Europe or Asia is related to the capability of the structure to resist wind action.
Gustave Alexandre Eiffel was famous for the Eiffel-type wind tunnel. He tried to conduct full-scale measurements of the response of the Eiffel tower under meteorological conditions including winds at the top of the completed 300m-high Eiffel Tower, the world’s tallest structure at that time (Davenport, 1975, p. 28).

Chen (2008) performed a frequency domain analysis of along-wind tall building response to transient non-stationary winds based on non-stationary random vibration theory.

Rofail (2008) has studied various available techniques for dealing with building forms in wind load scenarios. He has conducted a few case studies of unusual structures around the world and presented very useful data for engineers and researchers.

The researchers have mainly focused on wind’s characteristics, its properties and variations with respect to wind tunnel testing. The scholarly material has a huge gap in research about buildings’ overall behaviour under wind loads.

2.3.3 Review of academic work pertaining to earthquake actions

Seismic actions or earthquake forces are another deterrent in the design and construction of high-rise structures. The forces generated due to earthquakes could be very disastrous and hence special consideration needs to be given to structures in high seismic activity zones.

Hajjar (2002) has discussed in detail the components of composite systems such as columns, walls and connections to provide an insight into the future direction of composite construction with respect to seismic loadings.

A study undertaken by Choi & Park (2011) suggested a method of reducing the inter-story drifts of steel moment frames without changing the total structural weight. The authors used static linear analysis for equivalent static seismic loads on a 3-storey building.

The effect of component deterioration and ductile fracture on the seismic capacity of high-rise buildings was investigated by Lignos et al. (2011).

Han et al. (2009) has conducted shaking table tests on two building models of 30 stories that consisted of composite frames and RCC shear walls. The authors
evaluated the behaviour of mixed structures consisting of CFST columns under various earthquake records.

Su & Wong (2007) carried out an experimental study on three RCC wall specimens to study the effects of axial load ratio and confinement on their performance under artificial earthquake loads.

It is observed that most of the academic literature concentrates either on individual components of a structure under seismic loads or on characteristics and properties of earthquake loads. The research gap in investigating the overall serviceability and durability of composite buildings under seismic loadings requires to be addressed.

2.3.4 Lags in academic research

As discussed in sections 2.3.1, 2.3.2 and 2.3.3, although there are many research studies and academic publications available on the composite components of buildings, there is a scarcity of scholarly material on the overall behaviour of composite buildings.

Generalised theories and/or rules specific to composite buildings are scanty, while research, tests and analytical models for individual elements of composite structures are found in abundance. The structural designer has to go through a lengthy process of creating a whole model with most of the details, since there are no set procedures for finding out the requirements of outriggers and establishing the locations of these in a building.

It can be argued that every structure is different from every other, hence cannot be related. However, when it comes to regular everyday buildings, this gap is particularly noticeable. The procedure of optimisation is usually based on trial and error as well as on past experience. If a project is delayed or cancelled, the detailed work that has been done to establish the feasibility of the project could be wasted.

The aim of this thesis is to study the behaviour of composite buildings braced with belt-truss and outriggers under horizontal loadings through finite element modelling. A detailed parametric study has been carried out by varying heights, plans and number and placement of lateral bracings for commonly used building structures in Australia. This study will be beneficial in the formulation of generic principals or rules for normal/usual building structures which are covered by Australian standards.
It will also help engineers and structural designers in their everyday calculations of cost and material estimation without having to perform lengthy tasks and putting too much energy and time into the conceptual/primary stage of the project.

2.4 ONSET OF COMPOSITE CONSTRUCTION

2.4.1 Inception of composite construction

In the context of structural engineering, the term “composite construction” designates the combined use of structural steel and reinforced concrete in such a way that the resulting arrangement functions as a unique entity. The goal is to accomplish a higher level of performance than would have been the case had the two materials been used separately.

The start of composite construction can be traced back about 100 years. From the time of its inception, the efficiency of composite systems has been identified as a compelling way of augmenting structural performance. More and more steel structures are now designed compositely because of the effectiveness of RCC shear walls in lateral load resistance.

Nethercot (2004, p. 1) claims that the starting period of composite construction was 1894, when concrete encased beams were first used in a bridge in Iowa and a building in Pittsburgh.

The initial work on composite construction in Canada was traced back to 1922 by Chien & Ritchie (1993), when a series of tests was conducted on composite beams.

Zhong & Goode (2001) give an elaborate picture of composite construction in China with a focus on the design and detailing of concrete-filled steel tube columns.

The idea of composite construction of tall tubular buildings was first conceived and used by Fazlur Khan of Skidmore, Owings & Merrill (SOM) in the 1960s. This has paved the way for high-rise composite buildings like Petronas Towers and Jin Mao building. Super tall buildings such as the Burj Khalifa, the 151 storey Incheon Tower under construction in South Korea, and a proposed 1 km tower in Saudi Arabia, are all instigated by such indigenous thoughts (Mendis & Ngo, 2008, p.2).
Taranath (2012, p.96) stated that apart from economy of material and speed of construction, composite structures, due to being light weight, inflict less severe foundation conditions hence results in greater cost savings.

2.4.2 Case studies

Even though there is a lacking of academic work on overall behaviour of composite buildings, there are a few case studies specific to particular buildings or projects.

Figure 2.1. Capital Gate - Abu Dhabi

*Figure 2.1* shows view of City Gate tower Abu-Dhabi is retrieved from [http://www.e-architect.co.uk/dubai/capital_gate_abu_dhabi.htm](http://www.e-architect.co.uk/dubai/capital_gate_abu_dhabi.htm) (e-architect, 2010).

A case study was performed on Capital Gate Tower, Abu Dhabi (*Figure 2.1*) by Shofield (2012). The author described that the composite structure was built with a concrete core surrounded by steel trusses termed as “diarigid”. Steel beams supported the concrete composite floor and ran between external and internal vertical supports. Lateral wind actions were counteracted by the introduction of dense outriggers at the 17th mechanical floor, which connected the external frame to the central core.
Figure 2.2 illustrates elevation, brace connection and the plan of Goldin Finance 117 Tower, is retrieved from http://www.skyscrapercenter.com/tianjin/goldin-finance-117/73/ (The Skyscraper Centre, n.d.).

Tianjin Goldin Finance 117 tower (Figure 2.2) was studied by Peng et al. (2013). The authors wrote that the concrete core consisted of embedded steel sections and ran from the foundation to the top level. Mega columns provided at the four corners of the building were made up of internal inter-connected steel plates enclosed by external steel plates, hence forming a polygonal shape. The chambers within were filled with concrete and reinforcement was provided to satisfy axial, bending, buckling and torsional capacity. The mega columns were connected to each other with mega braces at the structure’s periphery. The lateral load resisting system comprised transfer trusses distributed every 12 to 15 floors; these connected the mega columns to the main core wall. The floor framing consist of a composite floor deck supported by steel beams.

2.5 PROFILED DECKING AS A PERMANENT FORM

Composite technology has a dual usage, that is, it can be used as a structural element as well as for permanent form work such as profiled decking.
Figure 2.3 exemplifying profile sheeting. Retrieved from http://www.tegral.com/index.php?page=Comflor (Tegral Comflor® Composite Flooring, n.d.).

Profiled steel decking consists of a corrugated steel sheet with an in-situ reinforced concrete topping (Figure 2.3). The decking acts as permanent formwork and also provides a shear bond with set concrete. Hence, when concrete gains strength, the two materials work together and the profiled sheeting acts as bottom reinforcement.

This type of formwork is extensively used throughout the world. In United Kingdom, 40% of various constructions use a composite slab system (Nagy et al, 1998).

The use of a composite slab is a remarkable advancement in the construction of high-rise buildings requiring open plan space. This has many benefits technically and economically. It serves the following main structural purposes:

- During the course of concreting, the metal decking supports the weight of the wet concrete and top reinforcement, together with temporary loads of construction.

- The decking acts ‘compositely’ with the concrete and serves as a bottom reinforcement in resisting sagging moments of the slab as occurs with a conventional reinforced concrete slab.

- The steel decking is also used to stabilise the beams against lateral torsional buckling during construction and to stabilise the building as a whole by acting as a diaphragm to transfer wind loads to the walls and columns.
• Robustness can be readily achieved by continuity between decking, reinforcement, concrete, secondary and primary elements.

The financial benefits are equally important in today’s competitive and enterprising construction industry. Rackham et al. (2009, p. 2) point out the commercial benefits of profiled metal decking: speed of construction, reduced weight of structure, easy transportation, shallow slab depth, sustainable construction and ease of service installation.

2.6 OVERVIEW OF FRAMING SYSTEM USED IN THIS STUDY

The framing system used in this research consists of RCC shear walls and reinforced concrete columns with embedded steel sections. Lateral stability is attained by tying the RCC wall with composite columns through belt-truss and outriggers (Figure 2.4). Belt-truss and outriggers are provided with the variations of single floor and double floors in different models. The belt-truss ties the peripheral columns of the building, while the outriggers engage them with the main or central shear wall. Therefore, exterior columns restrain the core wall from free rotation through the outrigger arms.

Gunel & Ilgin. (2007) described the outrigger system as an innovative and efficient structural system. The outrigger system comprises a central core, including either braced frames or shear walls, with horizontal outrigger trusses or girders connecting the core to the external columns.

Figure 2.4. Typical outrigger and belt-truss
Figure 2.4, illustrating a typical outrigger system. Retrieved from online material in http://www.structuremag.org/article.aspx?articleID=684 (Melchiorre, 2008).

Hal (1988) studied the deflection control on a two-dimensional model with the use of outriggers.

Kian et al. (2001) have analysed the efficiency of belt truss and outrigger in concrete high rise buildings.

Nair (1998) suggested a concept of a “virtual” outrigger system in which the stiffness of floor diaphragms could be utilised to transfer moment as a horizontal couple from the core to trusses or walls that are not connected directly to the core.

![Diagram of an outrigger system]

Figure 2.5. Basic view of, elevation and plan

The usefulness of belt-truss and outriggers is well-known, though there is always disagreement on the reduction of operational space at the outrigger level. This, however, can be curtailed by the use of diagonal cross bracing in line with the columns as well as the use of horizontal trusses that can be entrenched in a false ceiling. Typical outrigger arrangement is shown in Figure 2.5.
2.7 CONCLUSION

Composite construction is a brilliant and cost efficient solution developed in this era. In Australia, structural engineers readily employ this system to save time and material. It is popular in office and commercial buildings. For sufficient stiffness and efficient lateral load path, a system of outriggers and belt-trusses is normally coupled with composite construction.

Outriggers and belt-truss systems are in constant use in various high-rise developments, however, their use and provision is specific to a particular construction or building structure. Usually structural engineers have to conduct a rigorous analysis with a trial and error approach to find the number of steel braces required in a building and their placement along the height of building. Hence, certain generic rules and principles are needed that can help the structural designer to compute the requirement of bracings (i.e. core walls, outriggers, belt-truss etc.) based on structure height and plan dimensions (i.e. width and length). This would be helpful in the approximate judgment of various quantities and cost (i.e. material, labour cost, project time line etc.) without having to undertake a rigorous analysis.

Moreover, most research has been concerned with the components of composite structural systems of buildings, such as composite columns and composite beams. Seismic and wind actions are also investigated using analytical two-dimensional models that revolve around the characteristics and parametric properties. Therefore, this thesis aims to fill in the lack of academic research into the overall behaviour of buildings.
3.1 INTRODUCTION

Finite Element Analysis (FEA) is a numerical system for solving complex problems. In this method, structural elements are divided into finite elements and analysed for strain, stress, moments and shear etc. FEA has been embedded in engineering and other sciences and it is now essential in the solution of mathematical problems.

This research is conducted by analysing building prototypes through Finite Element Modelling (FEM). Strand7 (Strand7 R2.4.4, 2011) is chosen for research because of its availability in university and its popularity within the construction industry.

This chapter consist of descriptions of thesis models and criteria for the selection of height and plan layouts for these models. It also describes the calculation of transformed properties of composite elements, and the selection of properties for non-composite elements. The input of all these properties to the models and approximation of models with respect to Australian standards guidelines are explained. It covers the wind and seismic loads application in models, and finally, model validations are carried out to establish the prototypes’ reliability.

3.2 ANALYTICAL PROGRAMME AND SOLVERS FOR THESIS MODELS

3.2.1 Programme/software selection

Strand7 Release 2.4.4 (Figure 3.1) is selected for modelling and analysis of thesis prototypes. This choice is made primarily due to the availability of this program within university resources and also because of its popularity within Australia. Most Australian engineering design firms use this software on a day-to-day basis, and it is gaining fame in universities as well. A number of leading universities in Australia have already integrated Strand7 in their curriculum; for instance, Queensland University of Technology offers certain undergraduate courses that teach the basis of FEM through Strand7 (Strand7 R2.4.4, 2011).
Figure 3.1. Strand7 opening page

The program initiated in 1996 already has many buildings and real life structures in its credits, signifying the program’s reliability and integrity. For example: “the runner sculpture” placed on top of Sydney towers during 2000 Sydney Olympics (Figure 3.2), the optimisation of “Water Cube” in Beijing National Aquatic Centre for 2008 Beijing Olympics (Figure 3.3), and the roof design of Terminal 2E of the Charles de Gaulle Airport in Paris, France (Figure 3.4).

Figure 3.2. Runner sculpture, Sydney, Australia

Figure 3.2. shows Runner sculpture in Sydney, Australia is designed using Strand7 (Strand7 R2.4.4, 2011). Retrieved from http://www.flickr.com/photos/39551170@N02/5715508966/ (flickr, n.d.).
Figure 3.3. show an external view of Beijing National Aquatic Centre, China designed using Strand7 (Strand7 R2.4.4, 2011). Retrieved from http://architecture.about.com/od/greatbuildings/ig/Stadium-and-Arena-Pictures/Water-Cube.htm (About.com Architecture, n.d.).

Figure 3.4. shows an external view of Terminal 2E of Charles de Gaulle Airport, Paris, France, designed using Strand7 (Strand7 R2.4.4, 2011). Retrieved from http://structurae.net/structures/data/index.cfm?id=s0009234 (Structurae, n.d.).

3.2.2 Solvers used in thesis

Strand7 is suited for most engineering problems of almost every discipline of engineering and research. It has been equipped with wide range of solvers, modelling options and output styles. However, for every specific study or design, only those solvers which are applicable to that study are used. Accordingly, in this study, three types of solvers are used in the research and are described below.
3.2.2.1 Linear static solver

The linear static solver is based on the assumption that the structure’s behaviour is linear and applied forces are static. This is based on the elastic theory, that “element forces are linearly proportional to element deformation and when loading is removed the element will come back to its original shape”. Therefore, the model must follow “Hooke’s Law”. A load is static if its magnitude and direction do not vary with time.

Multiple load cases are treated in one solution in this solver. Combination load cases of primary loads are available through combining the results for primary load cases in the post-processor without running the linear static solver again. Displacements and results for all load combinations are calculated at the end of solution.

In the study, this solver is used for wind dynamic analysis and IHS analysis.
3.2.2.2 Natural frequency solver

The natural frequency solver (Figure 3.6) is used to calculate the natural/fundamental frequencies (or free vibration frequencies) and corresponding vibration modes of an un-damped structure.

The frequency solver offers flexibility in analysis through frequency shift, mass participation and strum check etc. The frequency shift helps avoid lower modes of vibration and processes kept towards the higher modes only. The strum check ensures that all the Eigen values have been converged successfully in the solution. Mass participation is used in SS analysis.

In the study, the frequency solver is used in both wind and seismic analysis of models.

3.2.2.3 Spectral response

The spectral response solver (Figure 3.7) computes the response of a structure exposed to a random dynamic loading. Two types of random dynamic loadings can be encountered: earthquake base excitation and general dynamic loads. In this study, earthquake base excitation is used for seismic analysis. The spectral curve is defined either as “a function of frequency” or “time period of vibration”.

Figure 3.6. Natural frequency solver
The solver calculates maximum model responses using two methods: CQC (Complete Quadratic Combination) and SRSS (Square Root of Sum of Square). The comparison of CQC and SRSS is not the objective of this study. Based on common design practices, SRSS is used for model solution.

In this study, the spectral response solver is used to find out displacements under SS loading.

### 3.3 LOADS ON MODELS

The primary loads or forces that dictate the design of most of the on-ground structures are gravity, wind and earthquake, although every structure does not analyse or design for all these forces. Structural analysis of a particular structure depends upon its location, situation, environmental conditions, architectural layout, height, width, usage, client requirements etc.

Loads acting on a multi-storey building can be broadly classified into static and dynamic loads and their derivatives, as represented by Figure 3.8.
3.3.1 Gravity loads

Loads and forces acting towards the centre of the earth are called gravity loads or gravitational loads or vertical loads. The basic figure of vertically acting loads is given in Figure 3.9, which also shows the load tributary area or load catchment area of the vertical support i.e. column. Gravity loads are calculated according to AS/NZS 1170.1 (2002) and consist of dead loads (G) and live loads (Q) (Figure 3.8).
The dead loads in models comprise structural self weight, partitions, and ceilings, air-conditioning ducts and services for office building scenarios, while live loads are predominantly human loads. The standard provides certain guides for live loads relative to occupancy. Live loads used in the models are for office occupancy.

3.3.2 Lateral loads

The loads or forces that act perpendicular to the vertical axis of building are called horizontal loads or lateral loads. These loads are discussed in chapters 4 and 5.

3.4 SELECTION OF PARAMETERS TO SATISFY THESIS OBJECTIVES

3.4.1 Aims

The choice of parameters integrated many factors; the main aim has been:

- To facilitate engineers in their everyday analysis and design work so that they do not have to do detailed calculations for basic structural prediction in the pre-design stages of a project.

- To study structures commonly found in the local environment, for instance, local builders prefer to construct office buildings with composite structure so that vast, uninterrupted rentable space can be achieved.
• To study structures in compliance with Australian general practices and Australian standards with the intention that everyday engineering works can be benefitted.

• To depict commonly used structural arrangement in prototypes that include floor to floor heights, building plans and building heights etc.

• To use properties of locally produced building materials and products such as floor sheeting from Lysaght Bondek (Lysaght Bondek, 2012), and steel section sizes from Australian Steel Institute (ASI, 2009) capacity tables.

• To gear up the research into the structural behaviour of composite buildings in conjunction with the outrigger system. Further, to add worthwhile material that will make a significant contribution of knowledge for incoming researchers and engineers in this field of engineering.

3.4.2 Parameters

This thesis is a comparative study between various building heights and different plan layouts. Hence, based on the above criteria and considering the local general practice, three types of parameters are selected:

• Model heights;
• Model plan layout;
• Belt-truss and outriggers variations.

3.4.2.1 Models heights selection

The following heights are selected based on the above considerations:

• 57-storey is 199.5 m high, given the storey heights as 3.5m. This is the maximum allowable height as per Australian Standards (AS/NZS 1170.2, 2011). This is chosen to study the effects of wind and seismic loads calculated according to Australian standards on a maximum given building elevation.

• 42-storey is 147.0 m high. This is most common type of multi-storey rise within the Australian urban environment. Many office and residential buildings are constructed around this height; hence this is an appropriate comparison with the 199.5 m tall model.
• 28-storey is nearly half the height of the 57 storey model, i.e. 98 m. This height is selected to establish a comparison and to find out the benefits (if any) of belt-truss and outriggers on such a short elevation.

3.4.2.2 Layout selection for models

The main object in layout selection is to allow maximum variation and maintain distinction. In all models, the Z-axis represents the vertical axis, whereas the X-axis and Y-axis are planner axes. The plan layouts selected are:

- Rectangular shape model
- Octagonal shape model;
- L-shaped model.

**Rectangular shaped**

The rectangular (Figure 3.10) shape is a common shape in Australian. Land demarcation is usually rectangular in most Australian municipalities; therefore developers tend to go for this shape of structure. Further, this shape has the appeal of having windows on both sides of the building, which yield higher rentable value. The layout has higher rigidity in one axis and less in the other; hence it is relevant to study the lateral load effects and frequency modes of this plan layout.

![Figure 3.10. Rectangular model elevation (full model and shear wall)](image)

**Octagonal shape**

This has equal plane dimensions (Figure 3.11) and hence can represent circular and square buildings. However; in square shapes there are re-entrant corners that produce swirling effects when subjected to wind actions. Since the wind dynamic
loads are calculated according to Australian standard (AS/NZS 1170.2, 2011), the need to counteract this effect is not significant unless the building exceeds the prescribed height and wind tunnel studies become essential. Therefore, the octagonal shape can stand for the square shape.

![Octagonal model elevation (full model and shear wall)](image)

**Figure 3.11. Octagonal model elevation (full model and shear wall)**

**L-shaped model**

This is selected to study an extended layout with double core walls in both of its arms (Figure 3.12). It is more massive than the other two shapes. The effects of lateral loads on this model are studied and compared with the other two less rigid models. The corner wall around the stair well and side walls are needed to stabilise the model and achieve the desired frequency mode shapes.

![L-Shaped model elevation (full model and shear wall)](image)

**Figure 3.12. L-Shaped model elevation (full model and shear wall)**
3.4.2.3 Outriggers provision in models

Belt-truss and outriggers are used as secondary bracings for lateral load resistance in conjunction with a primary bracing, that is, an RCC shear wall. The study’s main focus is the utilisation of belt-truss and outriggers in various ways in models and analysis of their outcomes.

Many shapes of truss system are available in the market; however, the crucial objective of this study is not the shape of the truss but its location. Therefore, a commonly used system of cross-bracing is adopted, as shown in Figure 3.13.

![Figure 3.13. Outriggers and belt truss](image)

The desirable structural system is one which has least obstructions, that is, fewer columns and outrigger levels and more rentable space. The floors with outriggers are mainly used for storage or as electrical and/or mechanical equipment rooms; hence they are usually not rentable and not desirable. Therefore, models are tried starting with one belt-truss and outriggers up to four truss levels. These levels are split along the height in a variety of double floor outriggers and single floor outriggers. The placement of truss levels are finalised based on the most effective places along the height of various models. These arrangements are kept the same in Rectangular, Octagonal and L-shaped models (Table 3.1).
Table 3.1

Model arrangements

<table>
<thead>
<tr>
<th>Model Title</th>
<th>Model arrangements</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>28-storey</strong></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
</tr>
<tr>
<td><strong>42-storey</strong></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
</tr>
<tr>
<td><strong>57-storey</strong></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
</tr>
<tr>
<td>57-2</td>
<td>Outrigger at top</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (level 38)</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (level 29)</td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, mid-height and 2/3rd height</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3rd height</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
</tr>
</tbody>
</table>

3.5 STRUCTURAL SETUP OF MODELS

To achieve the thesis objectives, finite element modelling of building prototypes is carried out within the limitations and scope of Australian standards: AS/NZS 1170.0 (2002), BCA (2012) and AS 4100 (1998). Prototypes are modelled as braced frame structures, i.e. additional structural elements are provided to resist lateral loads. These bracings are classified as primary bracings (i.e. RCC shear walls) and secondary bracings (i.e. belt-truss and outriggers).

Belt-trusses engage peripheral columns and outriggers connect these columns to RCC shear walls. Thus, horizontal loads on the structure get transferred from external columns to shear walls, which carry them to the foundation.

Li et al. (2010, p. 1) has emphasised that in steel-concrete hybrid structures, reinforced concrete shear walls with high lateral stiffness are usually selected to
confront horizontal loads originated by winds or earthquakes, while steel frames with
greater strength are generally designed to sustain the vertical loads. In addition,
hybrid structures can easily be tailored to large-span architectural space; therefore,
they are particularly attractive to real estate developers.

The elements used in prototypes are given in Table 3.2.

Table 3.2.
Models’ structural arrangement

<table>
<thead>
<tr>
<th>Element</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>Concrete deck with profiled sheeting</td>
</tr>
<tr>
<td>Main and Secondary Beams</td>
<td>Structural Steel Universal Beam sections.</td>
</tr>
<tr>
<td>Column</td>
<td>Steel WC and UC section encased in reinforced cement concrete</td>
</tr>
<tr>
<td>Core wall</td>
<td>Reinforced cement concrete (RCC)</td>
</tr>
<tr>
<td>Belt Truss and outriggers</td>
<td>Structural Steel UC sections</td>
</tr>
</tbody>
</table>

WC – Welded Column
UC - Universal Column

3.5.1 Prototype set-up in Strand7

Composite structural elements consist of slab and columns (*Figure 3.14*). Central core and side walls are RCC shear walls. Primary beams, secondary beams and belt-truss and outriggers are structural steel sections. In Strand7 models (Strand7 R2.4.4, 2011) columns, beams, belt-truss and outriggers are modelled as beam elements. Plate elements are assigned to composite slab and shear walls.
Figure 3.14. Slab and column composite sections

Figure 3.14 shows a section of composite slab and column, adopted from online material in http://www.steelconstruction.info/Composite_construction (SteelConstruction.info, n.d.).

3.5.1.1 Construction type

Simple construction is adopted for models based on the definition provided in Australian Standard (AS 4100, 1998, p. 35). Henceforth, rotation end releases are assigned to both ends of all primary and secondary beams to depict pin connections (Figure 3.15).
3.5.1.2 **Vertical support**

Fixed support is provided to the core and columns at the base (*Figure 3.16*). Analysis and design of the foundation is not within the scope of this thesis.
3.5.2 Transformed properties for composite elements

Equivalent transformed properties of slab and columns are used in three-dimensional models to mimic the maximum realistic behaviour of buildings under dynamic wind and seismic loads.

Modulus of Elasticity ($E$) and Density ($\gamma$) of composite elements are used to define composite members in Strand7 (Strand7 R2.4.4, 2011). Calculations of these properties are shown in Appendix A.

Composite mass contribution is included through density. For instance, the density of the RCC column is 2500 kg/m$^3$; however, with embedded I-section, the combined density becomes 2600 kg/m$^3$ (Figure 3.17). Similarly, the elastic modulus of 100MPa concrete is 42200.00 MPa and of structural steel is 200,000.00 MPa. The combined elastic modulus of composite column is higher than concrete and lower than a steel elastic modulus (Figure 3.17).

The transformed elastic modulus of composite section is given by Equation 3.3:

$$A_cE_c + A_{ST}E_s = A_gE_T$$  \hspace{1cm} 3.3

The transformed density of the composite section is given by Equation 3.4:

$$A_c\gamma_c + A_{ST}\gamma_s = A_g\gamma_T$$  \hspace{1cm} 3.4

Where:

<table>
<thead>
<tr>
<th>$A_g$</th>
<th>Gross area of section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c$</td>
<td>Area of concrete</td>
</tr>
<tr>
<td>$A_{ST}$</td>
<td>Area of steel</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Elastic modulus of concrete</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Elastic modulus of steel</td>
</tr>
<tr>
<td>$E_T$</td>
<td>Elastic modulus of transformed section</td>
</tr>
<tr>
<td>$\gamma_c$</td>
<td>Density of concrete</td>
</tr>
<tr>
<td>$\gamma_s$</td>
<td>Density of steel</td>
</tr>
<tr>
<td>$\gamma_T$</td>
<td>Density of transformed section</td>
</tr>
</tbody>
</table>
3.5.2.1 Composite column

For the composite column, various I-sections are selected from the Design Capacity Table for Structural Steel (ASI, 2009). In addition to I-sections, vertical bars along column edges/sides are included in the column capacity calculations. The steel to concrete ratio is kept well below 4%Ag (AS 3600, 2009) to accommodate lateral restraints, that is, stirrups.

Columns are divided into two main categories: internal columns with a load catchment area of 100 m$^2$/floor and edge columns with a load catchment area of 50 m$^2$/floor. The transformed properties of columns are summarised for 28 storeys, 42 storeys and 57-storey in Table 3.3, Table 3.4 and Table 3.5 respectively.

The sizes of columns are based on gravity loads; therefore a higher level column has a smaller cross-sectional area. In a 28-storey building, the maximum column size is 850 mm$^2$ (Table 3.3), whereas in a 57-storey building (Table 3.5) the maximum column size is 1250 mm$^2$.
### Table 3.3

**Column sizes of 28 storeys (98.0 m)**

<table>
<thead>
<tr>
<th>Levels (26 – 28)</th>
<th>( f'c ) (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>ET (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>( \gamma_T ) (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(26 – 28)</td>
<td>100</td>
<td>350</td>
<td>46992</td>
<td>250</td>
<td>46246</td>
</tr>
<tr>
<td>(21 – 25)</td>
<td>100</td>
<td>500</td>
<td>45664</td>
<td>2600</td>
<td>44665</td>
</tr>
<tr>
<td>(16 – 20)</td>
<td>100</td>
<td>650</td>
<td>46870</td>
<td>2600</td>
<td>47498</td>
</tr>
<tr>
<td>(11 – 15)</td>
<td>100</td>
<td>750</td>
<td>46607</td>
<td>2600</td>
<td>46246</td>
</tr>
<tr>
<td>(6 – 10)</td>
<td>100</td>
<td>850</td>
<td>45840</td>
<td>2600</td>
<td>47421</td>
</tr>
<tr>
<td>(1 – 5)</td>
<td>100</td>
<td>900</td>
<td>46855</td>
<td>2600</td>
<td>47054</td>
</tr>
</tbody>
</table>

*x-area = cross-sectional area of column in mm²*

### Table 3.4

**Column sizes of 42 storeys (147.0 m)**

<table>
<thead>
<tr>
<th>Levels (31 – 42)</th>
<th>( f'c ) (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>ET (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>( \gamma_T ) (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(31 – 42)</td>
<td>100</td>
<td>625</td>
<td>44940</td>
<td>250</td>
<td>44639</td>
</tr>
<tr>
<td>(26 – 30)</td>
<td>100</td>
<td>700</td>
<td>46797</td>
<td>2600</td>
<td>45340</td>
</tr>
<tr>
<td>(21 – 25)</td>
<td>100</td>
<td>800</td>
<td>47099</td>
<td>2600</td>
<td>46679</td>
</tr>
<tr>
<td>(16 – 20)</td>
<td>100</td>
<td>875</td>
<td>47930</td>
<td>2600</td>
<td>47864</td>
</tr>
<tr>
<td>(11 – 15)</td>
<td>100</td>
<td>950</td>
<td>47496</td>
<td>2600</td>
<td>47122</td>
</tr>
<tr>
<td>(6 – 10)</td>
<td>100</td>
<td>1025</td>
<td>47138</td>
<td>2600</td>
<td>46963</td>
</tr>
<tr>
<td>(1 – 5)</td>
<td>100</td>
<td>1075</td>
<td>47677</td>
<td>2600</td>
<td>46940</td>
</tr>
</tbody>
</table>

*x-area = cross-sectional area of column in mm²*

### Table 3.5

**Column sizes of 57-storeys (199.50 m)**

<table>
<thead>
<tr>
<th>Levels group</th>
<th>( f'c ) (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>ET (MPa)</th>
<th>( \gamma_T ) (kg/m³)</th>
<th>( \gamma_T ) (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(51 – 57)</td>
<td>100</td>
<td>500</td>
<td>45410</td>
<td>250</td>
<td>45423</td>
</tr>
<tr>
<td>(41 – 50)</td>
<td>100</td>
<td>700</td>
<td>47260</td>
<td>2600</td>
<td>45664</td>
</tr>
<tr>
<td>(31 – 40)</td>
<td>100</td>
<td>900</td>
<td>47109</td>
<td>2600</td>
<td>47723</td>
</tr>
<tr>
<td>(26 – 30)</td>
<td>100</td>
<td>950</td>
<td>47059</td>
<td>2600</td>
<td>47995</td>
</tr>
<tr>
<td>(21 – 25)</td>
<td>100</td>
<td>1000</td>
<td>47726</td>
<td>2600</td>
<td>48296</td>
</tr>
<tr>
<td>(16 – 20)</td>
<td>100</td>
<td>1050</td>
<td>47929</td>
<td>2600</td>
<td>47508</td>
</tr>
<tr>
<td>(11 – 15)</td>
<td>100</td>
<td>1150</td>
<td>46973</td>
<td>2600</td>
<td>46864</td>
</tr>
<tr>
<td>(6 – 10)</td>
<td>100</td>
<td>1200</td>
<td>47143</td>
<td>2600</td>
<td>47516</td>
</tr>
<tr>
<td>(1 – 5)</td>
<td>100</td>
<td>1250</td>
<td>46755</td>
<td>2600</td>
<td>46940</td>
</tr>
</tbody>
</table>

*x-area = cross-sectional area of column in mm²*
3.5.2.2 Composite slab

Composite slabs consist of corrugated profiled sheeting with concrete topping (Figure 3.14). The overall depth of slab is selected as 120 mm (Lysaght Bondek, 2012) for a 2.5m span length. The cross-sectional area, steel modulus and density of sheeting are extracted from Bondek’s manual (Lysaght Bondek, 2012). The spread sheet is formulated and formulas are entered to calculate composite slab properties (Figure 3.18). Floor loads are kept the same throughout the building because the main focus of the thesis is to study models under lateral loads for serviceability.

![Composite slab (abstract from Appendix C)](image)

3.5.3 Core wall/shear wall arrangements:

Model plans must satisfy AS 3600 (2009) and BCA (2012) in order to achieve the thesis objectives. The process of complying with Australian standards is tedious and repetitive and involved many “model runs” and “re-runs”, to:

- Satisfy minimum thickness for FRL (Fire Rating level);
- Comply with the access and egress requirements;
- Attain certain shear wall arrangements so the first two natural frequency modes represent the translational modes of structural vibrations.

The first two goals are achieved by placing the lift shaft and stairs at appropriate positions in the layout. Natural frequency mode shapes of building are governed by the core wall position in the layout, whereas the values of natural frequencies are controlled by wall thicknesses. Increasing wall thicknesses does not
help to shift the centre of rigidity of the structure, and hence does not help to change mode shapes. Therefore, the shear wall position in the layout is adjusted and re-adjusted to achieve required mode shapes.

The final shear wall arrangement around the lift core is shown in Figure 3.19. The L-shaped layout has two core walls around two lift shafts placed at two arms of the building, while the rectangular and octagonal layouts are provided with one lift shaft in the centre.

![Typical for "L Shaped" & "Rectangular" Model](image1)

![Typical for "Octagonal" Model](image2)

Figure 3.19. Core layouts

### 3.5.4 Structural steel elements

Steel sections are provided for main (primary) beams, secondary beams and steel bracing members (belt-truss and outriggers). Secondary beams are typically 10 m spans with 2.5 m centre to centre distance, and are supported on main/primary beams. Main/primary beams are provided at 10 m spacing and typically span 10 m. These sections are selected based on the OneSteel-DN3 (2005) guidelines and are listed in Properties of these sections are directly input from Strand7 (Strand7 R2.4.4, 2011) build-in library of programme.

Table 3.6. Properties of these sections are directly input from Strand7 (Strand7 R2.4.4, 2011) build-in library of programme.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Sections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main/primary beam (PB1)</td>
<td>610UB125</td>
</tr>
<tr>
<td>Main/primary beam (PB2) &gt; 10 m span</td>
<td>700WB173</td>
</tr>
</tbody>
</table>
3.6 ANALYSIS OF RESULTS

Analysis of wind and seismic effects show deflection, storey drift and natural frequencies. Dynamic effects are injected in the lateral loads calculations through natural frequency.

- **Deflection** ($\Delta$) or displacement is the deviation of the whole structure or structural element from its neutral position under an applied load and is measured in “mm”.

- **Storey Drift** ($\delta$) or inter-storey displacement is the lateral drift of a level relative to the level below in a multistorey structure and is measured in “mm”.

- **Frequency** ($f$) is the oscillation of any object about its neutral (central) position. In structural engineering the number of complete cycles of the to-and-fro motion of a building about its neutral axis is called its frequency. Frequency is a reciprocal of the time period, i.e. $f = 1/T$ and is measured in “Hz”.

3.7 OPTIMISATION PROCEDURE

To achieve a structural arrangement that satisfies frequency criteria and deflection limits of the relevant standards is a repetitive task and a “trial and error” procedure.

Jayachandran (p. 5, 2009) wrote that overall optimisation of a tall building frame has been complex and time consuming.

To comply with Australian standards, models are optimised for wind and seismic analyses. Some of the optimisation steps are common for both loadings; these are:

- Input of minimum prescribed wall thickness, column sizes and slab and beam properties for first run of model;
Structural Modelling

- Self-weight reactions and model mass are extracted from the programme and compared with manually calculated values as an initial model validity check;
- The first run is “natural frequency analysis” that gives the fundamental frequency of vibration of a structure. Models are run and re-run many times. For each solver cycle, shear wall positions and thicknesses are adjusted until the desired mode shapes are achieved.

**Wind analysis**

In addition to the above steps, optimisation for wind analysis includes the following:

- Acquired frequency is used to calculate dynamic cyclonic wind loads, which consist of along-wind and crosswind responses;
- Along-wind and crosswind responses are then applied in the directions of first mode and second mode of frequency respectively in the model (Strand7 R2.4.4, 2011);
- Australian Standards advocate using along-wind and crosswind responses simultaneously on a structure. Load combinations for these are given in Table 3.7.

<table>
<thead>
<tr>
<th>Load combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cases</td>
</tr>
<tr>
<td>Along-wind (Y-axis)</td>
</tr>
<tr>
<td>Crosswind (X-axis)</td>
</tr>
</tbody>
</table>

**Seismic analysis**

The steps for earthquake analysis are followed after the common steps. The load combination in Table 3.8 is common for HS and SS loadings.

<table>
<thead>
<tr>
<th>Seismic load cases</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Combinations</td>
</tr>
<tr>
<td>Cases</td>
</tr>
<tr>
<td>Force X-dir.</td>
</tr>
<tr>
<td>Force Y-dir.</td>
</tr>
</tbody>
</table>
Horizontal design response spectrum analysis (HS)

The HS analysis is performed as:

- The frequency for the first mode of vibration is used to calculate horizontal design action coefficient \( C_d(T) \);
- Factor \( \beta \) for HS load is then calculated as given in Strand7 Web notes (2011) as in Equation 3.5:

\[
\beta = \left[ k_p Z C_h(T_1) \frac{S_p}{\mu} \right]  \tag{3.5}
\]

This is used to generate horizontal shear in X and Y directions based on structural self-weight and non-structural mass already provided during modelling.

Site-specific design response spectra analysis (SS)

The calculation of SS analysis is based on two variables, i.e. “site sub-soil profile” (given in chapter 5) and “modal mass participation”;

- Modal mass is obtained by natural frequency analysis. The model is run up to ten frequency modes to achieve desirable mass participation.
- Site-soil profile is given by the normalised response spectra in the form of a graph, further described in chapter 5.
- The factor for SS loading is given in web note (Strand7 Webnotes, 2011) as in Equation 3.6:

\[
\text{Factor} = \left[ k_p Z \times \frac{S_p}{\mu} \right]  \tag{3.6}
\]

- The above values are then input in strand7 (Strand7 R2.4.4, 2011) for SS analysis.
3.8 MODELLING VALIDATION

Bhavikatti (2004, p. 7) stressed that a designer must get the feel of structure and structural behaviour and only use a program to get numerical results. He further stated that no matter what the reliability of a computer programme is, real structural behaviour could not be dictated or controlled by the computer programme.

Robustness and connectivity of models are verified by comparing values of internal and edge column reactions obtained from Strand7 (Strand7 R2.4.4, 2011) with manually calculated vertical loads. Further, the results of lateral forces and moments are also compared to satisfy the reliability and accuracy of models. These calculations are attached as Appendix A.

28-storey Octagonal model (98.0 m)

The largest difference is seen in along-wind and crosswind base shear (Table 3.9) of the octagonal 28-storey model. This is because the diagonal boundary of the layout is a 45 degree angle. The wind forces are entered as beam linear force in the model. Hence, the programme calculates these forces by the boundary angle (i.e. at 45 degrees), while a straight line is assumed in hand calculations.

<table>
<thead>
<tr>
<th>Items</th>
<th>Manual Cals</th>
<th>Strand7</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior column load (kN)</td>
<td>5392</td>
<td>5192</td>
<td>3.7%</td>
</tr>
<tr>
<td>Interior column load (kN)</td>
<td>9770</td>
<td>10008</td>
<td>2.44%</td>
</tr>
<tr>
<td>Base shear – along-wind response (kN)</td>
<td>798422</td>
<td>733977</td>
<td>8.07%</td>
</tr>
<tr>
<td>Base Shear - crosswind (kN)</td>
<td>634895</td>
<td>711264</td>
<td>10.73%</td>
</tr>
<tr>
<td>Base Shear – horizontal design response spectrum</td>
<td>1230</td>
<td>1299</td>
<td>5.311%</td>
</tr>
<tr>
<td>Overturning moment- horizontal design response spectrum</td>
<td>91439</td>
<td>92993</td>
<td>1.672%</td>
</tr>
</tbody>
</table>

42-Storey L-Shaped model (147.0 m)

In Table 3.10, base shear due to along-wind and crosswind responses have the highest differences. Manually calculated values are higher because linear force cannot be assigned to the entire length due to the presence of RCC walls around the stair well (at the corners), whereas total length and width are considered in manual calculations. Yet the difference is within acceptable limits of 5% to 10% as an adopted general practice of validation.
Table 3.10

Summary of modelling validation for 4-Storey L-Shaped model (147.0 m)

<table>
<thead>
<tr>
<th>Items</th>
<th>Manual Cals</th>
<th>Strand7</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior column load (kN)</td>
<td>8315</td>
<td>8363</td>
<td>0.57%</td>
</tr>
<tr>
<td>Interior column load (kN)</td>
<td>16929</td>
<td>16631</td>
<td>1.792%</td>
</tr>
<tr>
<td>Base shear – along-wind response (kN)</td>
<td>2068261</td>
<td>2177840</td>
<td>5.30%</td>
</tr>
<tr>
<td>Base Shear - crosswind (kN)</td>
<td>1983308</td>
<td>1824130</td>
<td>8.03%</td>
</tr>
<tr>
<td>Base Shear – horizontal design response spectrum</td>
<td>1121</td>
<td>1121</td>
<td>0.001%</td>
</tr>
<tr>
<td>Overturning moment- horizontal design response spectrum</td>
<td>124181</td>
<td>123887</td>
<td>0.236%</td>
</tr>
</tbody>
</table>

57-Storey rectangular model (199.50 m)

All the values in the rectangular model, as given in Table 3.11, are very close and the differences are minimal. The base shear of the along-wind response has the highest percentage of difference among all the comparisons in Table 3.11, while the base shear due to HS loads has the least difference.

Table 3.11

Summary of modelling validation for 57-Storey rectangular model (199.50 m)

<table>
<thead>
<tr>
<th>Items</th>
<th>Manual Cals</th>
<th>Strand7</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exterior column load (kN)</td>
<td>11609</td>
<td>11816</td>
<td>1.866%</td>
</tr>
<tr>
<td>Interior column load (kN)</td>
<td>24259</td>
<td>24379</td>
<td>0.544%</td>
</tr>
<tr>
<td>Base shear – along-wind response (kN)</td>
<td>4398990</td>
<td>4282334</td>
<td>2.65%</td>
</tr>
<tr>
<td>Base Shear - crosswind (kN)</td>
<td>3751029</td>
<td>3756480</td>
<td>0.15%</td>
</tr>
<tr>
<td>Base Shear – horizontal design response spectrum</td>
<td>1301</td>
<td>1301</td>
<td>0.001%</td>
</tr>
<tr>
<td>Overturning moment- horizontal design response spectrum</td>
<td>186705</td>
<td>186364</td>
<td>0.1828%</td>
</tr>
</tbody>
</table>

Difference = \{(Manual load – strand7 output)/ Manual Load\} x 100
3.9 CONCLUSION

The chapter has presented successful, stable and robust models of Strand7 (Strand7 R2.4.4, 2011). The process involved calculations of transformed properties of slab and column and their application in FEM. The optimisation of shear walls was carried out to achieve desired mode shapes for fundamental frequency. Robustness of the model was ascertained through comparison of various manually calculated and programme-generated outputs. The validation percentages were within the acceptable limits of 5-10%, which was also indicative of correct input parameters in the models.
Chapter 4: Wind Actions on Buildings

4.1 INTRODUCTION

This chapter outlines the derivation, orientation and application of wind actions on office prototypes developed in finite element software (Strand7 R2.4.4, 2011). These forces are defined and their computation is carried out with respect to AS/NZS Standard 1170.2 (2011). Selection of various variables and multipliers is performed with regard to the thesis topic and their choice is then justified. The application of wind linear force in FEM is given in detail.

The results contained in this chapter provide comparison between deflections and frequencies of various models. Altogether, fifty seven-models are run. The three plan layouts (i.e. Rectangular, Octagonal and L-shaped) are analysed and studied for three different heights (i.e. 98m, 147m and 199.5m). All these models are solved for a variety of belt-truss and outrigger options. Findings and irregular trends are discussed with the help of graphs and tables. The conclusion and results are presented at the end of chapter.

In the following paragraphs, the terms “truss” and “outriggers” will be used interchangeably and these have the same meanings i.e. “belt-truss and outriggers”.

4.2 WIND

Wind is perceptible natural movement of air; its flow can be suave like a zephyr or can be haphazard and tempestuous. Structural engineering translates wind as a natural phenomenon that puts forward an obtrusive force on buildings.

Taranath (2010, p. 255) states that wind is the term used for air in motion and is usually applied to the natural horizontal motion of the atmosphere.

Air flow is three-dimensional; it has one vertical and two horizontal components. In multi-storey building design, vertical air flow is of less significance than horizontal air flow. The vertical air pressure is counteracted by the weight of a building and hence is not a peril. Therefore, the terms wind action, wind force, wind load, wind speed, wind velocity and wind pressure all correspond to the horizontal component of air flow.
4.3 AIM AND SCOPE OF MODELLING FOR WIND LOAD ANALYSIS

4.3.1 Aims

Wind actions are calculated according to the guidelines of wind standard (AS/NZS 1170.2, 2011);

- To assist civil/structural engineers in Australia;
- To add useful and valuable research material to the selected topic;
- To offer new directions in research to upcoming scientist and researchers.

4.3.2 Scope

Principles and statistics provided in AS/NZS 1170.2 (2011) are based on wind tunnel testing, field measurements of various locations in Australia and New Zealand and established fluid mechanics rules. The methods and procedures outlined in the standard are sufficient to achieve realistic wind actions in standard situations. However, there are certain limitations relating to structural height and fundamental frequency. In addition, structures like lattice towers, offshore structures and bridges are outside the scope of the wind standard.

4.3.3 Compliance with AS/NZS 1170.2 (2011)

Three different plans (chapter 3) rectangular, octagonal and L-shaped are generated for the maximum height of 199.5 m (i.e. 57-storeys) which satisfies the requirement of clause 1.1 of the standard (AS/NZS 1170.2, 2011).

Plan dimensions of the rectangular model are 30 m and 80 m (Figure 4.1); plan dimensions of the L-shaped model are 60 m and 80 m (Figure 4.2); and plan dimensions of the octagonal model are 60 m in each direction (Figure 4.3); these are also in compliance with AS/NZS 1170.2 (2011) and BCA (2012).

AS/NZS 1170.2 (2011) is only applicable for a frequency range of 0.2Hz to 1.0Hz. The frequency is directly related to structural stiffness, as given in equation 4.1:

$$f_n = \frac{1}{2\pi} \sqrt{k/m} \quad Hz$$

Here;

$$f_n = \text{natural/fundamental frequency}$$
\[ k = \text{stiffness (N/m)} \]
\[ m = \text{mass (kg)} \]
\[ \pi = 3.14159265359 \]

If mass is kept constant than frequency is directly proportional to the square root of stiffness. Hence, the higher the stiffness the greater would be the frequency.

Figure 4.1. Force on rectangular layout

Figure 4.2. Force on L-shaped layout
4.4 DETERMINATION OF WIND ACTION TYPE

Wind actions on any structure or structural component can be “static” or “dynamic” as classified by standard (AS/NZS 1170.2, 2011). The decision as to the type of actions to be applied on any structure or structural component depends on variables such as frequency, dimensions and location.

A steady flow of wind exerts “static forces” while turbulent wind applies “dynamic forces” to structures. When a wind gust touches its maximum value and dies out in a time much longer than the vibration period of the building, the wind action is considered as static. Whereas if a wind gust attains its peak value and dies down in a shorter time than the period of the building, its effects are dynamic.

As per AS/NZS 1170.2 (2011), if a structure has a frequency more than 1 Hz then it is analysed and designed for static wind loads. Stocky structures fall in this category, while lean or tall structures usually have frequencies less than 1Hz. Hence in tall structures, dynamic loadings imposed by the wind are critical.
The models in this study have a frequency below 1 Hz, therefore dynamic wind loads are applied in FEM (Strand7 R2.4.4, 2011).

4.5 CHOICE AND CALCULATIONS OF WIND VARIABLES FOR MODELS

Detailed wind calculations are attached in Appendix B. The summary is given in Figure 4.4. Selections of various variables are given in subsequent paragraphs.

4.5.1 Selection of wind region

The map of Australia (see Figure 4.5) is divided into 9 wind regions based on “wind speed”. These regions ranges from A (1 - 7) to D and W. Cyclonic wind regions are allocated region C and region D, while region W constitutes only certain parts of New Zealand.

Region C is chosen for this study, which represent cyclonic wind areas. Region C extends from West to North, encompassing the regional towns of Broome, Wyndham and extending to Darwin. Then this region extends towards the east along the green belt/coast line and covers major regional cities, i.e. Cairns, Townville, Mackay, Rockhampton and Bundaberg.
With the escalating metallurgical and mining industry, the areas covered by region C are becoming ebullient and animated. The municipalities near the coast in region C are frequented by substantial passers-by as well as long term settlers. Progression and expansion is underway with the emergence of city centres, shopping malls, hospitals, recreational facilities and amenity yards. These could be the future cities of Australia with high-rise and multi-storeys. In short, Region C not only represents cyclonic wind actions but also extends to a larger part of coastline covering prospective cities with the potential of changing into metropolises.

Wind forces are calculated in concurrence to region C specifications of Australian standards and applied to commonly use office building prototypes.

**Figure 4.5. Wind Regions of Australia**

*Figure 4.5 shows wind regions of Australia; reproduced from AS/NZS 1170.2 (2011, p. 17).*

### 4.5.2 Selection of terrain category for models

Region C generally covers small townships and rural housing. Therefore, obstructions are usually scattered, with dispersed houses and scattered yards,
warehouses or factories etc. Therefore terrain category 2 is selected for wind pressure calculations, as given in standard (AS/NZS 1170.2, 2011).

4.5.3 Choice of site wind speed

The site wind speed is independent of type and shape of structure. It is calculated by multiplying regional wind speed ($V_R$) given in Table 3.1 of standard (AS/NZS 1170.2, 2011) by various factors as in equation 4.2;

$$V_{sit,\beta} = V_R M_{z,cat} (M_s \times M_t \times M_d) \quad m/s$$ 4.2

The regional speed ($V_R$) is based on the real gust factor. Since this study is focused on the serviceability performance of a structure, a return period of 25 years is taken as recommended by standard (AS/NZS 1170.0, 2002). A return period of 25 years represents an annual probability of exceedance of 1/25 (i.e. $P = 0.04$).

For regions B and C and D no direction is defined, and one value of “$M_d$” is given for all directions in clause 3.3.2 of AS/NZS 1170.2 (2011).

Region C and terrain category 2 specify no significant shielding, hence a maximum value of “$M_s = 1$” is considered for this study.

Region C constitutes a large area of the Australian boundary (Figure 4.5). The exact location of a site cannot be established within this study. Therefore; topographic multiplier is taken as “$M_t = 1$”.

The values of “$M_{z,cat}$” (i.e. terrain and height multiplier) are given in Table 4.1 of standard (AS/NZS 1170.2, 2011) for structures up to 200 m high. As the height of a structure increases, the values of $M_{z,cat}$ also increase. For intermediate values, a linear interpolation is recommended in the standard (AS/NZS 1170.2, 2011).

4.5.4 Determination of design wind speed

Design wind speed ($V_{des,\beta}$) represents directional consideration and height variations of a structure. Models are hypothetical exemplifications of office buildings; therefore the orientation of a structure on site with respect to wind direction cannot be attained. Therefore a non-directional method is used in which the maximum specific directional speed is applied to all directions (as acceptable in the standard). Hence, it is assumed that:
\[ V_{\text{sit,}\beta} = V_{\text{des,}\phi} \]

**4.5.5 Determination of design wind pressure**

The design wind pressure is given in equation 4.3 as:

\[ p_z = (0.5 \rho_{\text{air}}) \left[ V_{\text{des,}\phi} \right]^2 C_{\text{fig}} C_{\text{dyn}} \quad \text{kPa} \]

Equation 4.3

Standard (AS/NZS 1170.2, 2011) recommends that air density is 1.2 kg/m\(^3\) for 21°C at typical ground level.

The aerodynamic shape factor \( C_{\text{fig}} \) accounts for building shape, openings in enclosed buildings, and friction drag forces generated due to the wind. Wind action is the sum of internal and external pressures. Generally, in the case of tall, enclosed buildings, the most severe combination of internal and external pressures is adopted for analysis and design. In this study, the same practice is espoused, and a worse combination of \( C_{\text{fig}} \) is selected, as given in section 5 of standard (AS/NZS 1170.2, 2011) as represented in equation 4.4.

\[ C_{\text{fig}} = (\text{Internal Coefficient}) + (\text{External Coefficient}) \]

Equation 4.4

Where:

\[ C_{\text{fig},e} = C_{\text{p,i}} K_{c,i} \]

\[ C_{\text{p,i}} = \text{Internal component selected for “all walls are equally permeable”}. \]

\[ K_{c,i} = \text{combination factor applied to internal pressure taken as 1 for whole building structure}. \]

\[ C_{\text{fig},e} = C_{\text{p,e}} K_a K_{c,e} K_l, K_p \]

\[ C_{\text{fig},e} = \text{External component selected for structures having } h > 25\text{m}. \]

Factors \( K_a, K_{c,e}, K_l \) and \( K_p \) are effective if wind loads are required on individual structural and non-structural components of a building such as steel rafters, purlins, columns or claddings etc. However, in determining pressure on the overall structure the usual practice is to consider the maximum value of these coefficients (=1.0) in equation 4.4.
The dynamic response factor ($C_{dyn}$) takes into account the dynamic wind effects of wind according to chapter 6 of the standard (AS/NZS 1170.2, 2011).

4.5.6 Determination of dynamic response factor ($C_{dyn}$)

There are two components of dynamic response, along-wind and crosswind responses, as shown in Figure 4.6.

![Figure 4.6. Along-wind and crosswind on structure](image)

4.5.6.1 Along-wind direction for models

Wind is critical in the less stiff direction of a structure with 1st frequency mode. In Rectangular and L-shaped models, the weaker direction is along the Y-axis, which is perpendicular to the long side of the plan, as shown in Figure 4.1 and Figure 4.2. The stiffness of the octagonal model (Figure 4.3) is dictated by the central core and side walls. The Y-axis is also the weak axis in the octagonal model, accordingly the direction of wind is taken parallel to Y-axis. $C_{dyn}$ for along-wind response of models is calculated using equation 4.5.

\[
C_{dyn} = \frac{1 + 2I_h \sqrt{g_v^2 B_s + \frac{H_s g_R^2 S E_t}{\zeta}}}{(1 + 2g_v I_h)}
\]

In the above equation, the term “$g_v^2 B_s$” signifies the structural responses due to background dynamic forces and “$H_s g_R^2 S E_t$” represents the estimate of “amplified contribution response at natural frequency”.
4.5.6.2 Crosswind response

Crosswind is perpendicular to wind direction and in the direction of the second fundamental frequency mode. Hence crosswind response is a transverse component of wind velocity (Figure 4.6) and is given in equation 4.6.

\[ w_{eq}(z) = 0.5 \rho_{air} [V_{des,b}]^2 d \left( C_{fig} C_{dyn} \right) N/m \] 4.6

Stiffness is the main contributor of along-wind and crosswind pressure. Since models have higher stiffness in transverse directions, crosswind loads usually have larger values than along-wind pressure.

4.6 CALCULATION OF WIND PRESSURE

Wind loads are calculated with the help of an Excel spread sheet (Figure 4.7) for three different model heights and applied as horizontal linear pressure (kN/m) on edge beams in Y-dir. and X-dir. Wind pressure increases towards the top of a structure.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Height (m)</th>
<th>Width/ breadth (m)</th>
<th>M (kN.m)</th>
<th>Vz (m/s)</th>
<th>Px (kN/m²)</th>
<th>Horizontal Distributed Force (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>98</td>
<td>60</td>
<td>1.233</td>
<td>58.19</td>
<td>2.24</td>
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<tr>
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<td>60</td>
<td>1.236</td>
<td>58.09</td>
<td>2.22</td>
<td>7.81</td>
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<tr>
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<td>91</td>
<td>60</td>
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<td>57.93</td>
<td>2.22</td>
<td>7.77</td>
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<td>57.81</td>
<td>2.21</td>
<td>7.73</td>
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<td>2.19</td>
<td>7.66</td>
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<td>1.211</td>
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<td>60</td>
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<td>56.96</td>
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<td>7.51</td>
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<td>56.43</td>
<td>2.11</td>
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<td>60</td>
<td>1.169</td>
<td>55.93</td>
<td>2.07</td>
<td>7.24</td>
</tr>
<tr>
<td>15</td>
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<td>60</td>
<td>1.148</td>
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<td>2.05</td>
<td>7.16</td>
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<td>49</td>
<td>60</td>
<td>1.178</td>
<td>55.17</td>
<td>2.03</td>
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</tr>
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<td>60</td>
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<td>2.00</td>
<td>7.01</td>
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<td>60</td>
<td>1.164</td>
<td>54.71</td>
<td>1.98</td>
<td>6.92</td>
</tr>
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<td>38.5</td>
<td>60</td>
<td>1.154</td>
<td>54.24</td>
<td>1.94</td>
<td>6.81</td>
</tr>
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<td>35</td>
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<td>1.144</td>
<td>53.58</td>
<td>1.90</td>
<td>6.64</td>
</tr>
<tr>
<td>9</td>
<td>31.5</td>
<td>60</td>
<td>1.126</td>
<td>52.82</td>
<td>1.85</td>
<td>6.48</td>
</tr>
<tr>
<td>8</td>
<td>28</td>
<td>60</td>
<td>1.112</td>
<td>52.26</td>
<td>1.81</td>
<td>6.32</td>
</tr>
<tr>
<td>7</td>
<td>24.5</td>
<td>60</td>
<td>1.093</td>
<td>51.61</td>
<td>1.78</td>
<td>6.16</td>
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<td>21</td>
<td>60</td>
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<td>50.95</td>
<td>1.72</td>
<td>6.00</td>
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<tr>
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<td>17.5</td>
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<td>1.005</td>
<td>50.26</td>
<td>1.68</td>
<td>5.80</td>
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<td>60</td>
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<td>49.88</td>
<td>1.58</td>
<td>5.63</td>
</tr>
<tr>
<td>3</td>
<td>10.5</td>
<td>60</td>
<td>1.005</td>
<td>47.24</td>
<td>1.47</td>
<td>5.16</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>60</td>
<td>0.946</td>
<td>44.46</td>
<td>1.31</td>
<td>4.57</td>
</tr>
<tr>
<td>1</td>
<td>3.5</td>
<td>60</td>
<td>0.91</td>
<td>42.77</td>
<td>1.21</td>
<td>4.23</td>
</tr>
</tbody>
</table>

*floor 0*: ground

Figure 4.7. Abstract from worksheet: along-wind force calculation.
4.7 APPLICATION OF WIND LOADS ON MODELS

The most popular way to apply wind loads on finite element models is as a linear force on elements or members. The general equation of calculating $p_z$ (wind pressure normal to surface in kN/m$^2$) is given in equation 4.7.

$$p_z = 0.000683 \times [V_z \times M_{Z,cat}]^2 \text{ kN/m}^2$$  \hspace{1cm} 4.7

The number 0.000683 represents wind multipliers, as explained in section 4.5, and evaluated in Appendix B. The only variable in equation 4.7 is $M_{Z,cat}$. The “$p_z$” is multiplied by a half storey height above and a half storey height below to get the linear pressure in kN/m on a particular level (Figure 4.8). This pressure is then applied to horizontal beam members in Strand7 (Strand7 R2.4.4, 2011) in the appropriate direction. The application of pressure using this approach is less complicated and checks could be performed with certainty.

![Figure 4.8. Linear wind pressure on FEM Model](image)

4.8 WIND LOAD INPUT IN MODELS

Strand7 R2.4.4 (2012) provides the facility of force provision in local axes (i.e. x, y and z) and global axes (X, Y and Z) (Figure 4.9).
Wind action is considered a global phenomenon that is, acting on the overall structure, because the target is to examine the overall structural serviceability performance. Therefore, these forces are applied as “Global Pressure” in kN/m by selecting horizontal beam members on each level in Y-dir. and X-dir. for along-wind and crosswind pressure respectively (Figure 4.10).

Linear wind force along the +ve Y-axis of the rectangular (partial) model is applied on edge beams as shown in Figure 4.10. These edge beams are designated as main beams in the model.

Figure 4.10. Along-wind linear force on Rectangular Model (partial model)

Figure 4.11 shows the octagonal model. The load is in global axes, therefore linear arrows are seen on the diagonal beams.
Linear force on main beams in +ve Y-direction is illustrated in Figure 4.11 in the L-shaped model.
4.9 FEM ANALYSIS AND OUTPUT

4.9.1 Model analysis

The load combination used follows the guidelines of standard (AS/NZS 1170.1, 2002) and is given as equation 4.8;

\[
Load\ Combination = 0.5G + 1.0 W_x + 1.0 W_y
\]

In the above equation G stands for gravity loads, whereas \( W_x \) and \( W_y \) are wind in along-wind and crosswind direction respectively. The steps for wind analysis include:

- Analysis of three model types through “Natural Frequency” solver.
- The first two linear modes are established. The third torsional mode is not used in this study.
- The two linear modes are then used in an Excel sheet to calculate along-wind and crosswind actions on office prototypes.
- Thereafter these forces are applied to the model with gravity loads.
- Finally the models are run through “Linear Static” solver to attain results.

These models are, however, solved for many belt-truss and outrigger options as listed in Table 3.2, Table 4.2 and Table 4.3.

For each plan layout, nineteen models are run, which amounts to fifty-seven models altogether.

4.9.2 Model output

To perform the serviceability performance review of the models’ values of frequency, deflections and storey drifts in X-dir. and Y-dir. are extracted from the analysis results and listed in Table 3.2, Table 4.2 and Table 4.3.
Table 4.1

Results for rectangular models

<table>
<thead>
<tr>
<th>Rectangular Model</th>
<th>Model arrangements</th>
<th>Frequency</th>
<th>Deflection at top</th>
<th>max. Storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mode 1 (Y-Dir)</td>
<td>Mode 2 (X-Dir)</td>
<td>DX</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.4253</td>
<td>0.4532</td>
<td>91</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
<td>0.4466</td>
<td>0.4702</td>
<td>81</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>0.4492</td>
<td>0.4857</td>
<td>78</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
<td>0.2160</td>
<td>0.2466</td>
<td>316</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
<td>0.2314</td>
<td>0.2590</td>
<td>278</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>0.2322</td>
<td>0.2677</td>
<td>269</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>0.2456</td>
<td>0.2788</td>
<td>241</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>0.2428</td>
<td>0.2687</td>
<td>252</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>0.2475</td>
<td>0.2861</td>
<td>237</td>
</tr>
<tr>
<td>57-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
<td>0.1682</td>
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</tr>
<tr>
<td>57-2</td>
<td>Outrigger at top</td>
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<td>0.2140</td>
<td>326</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (lvl 38)</td>
<td>0.1761</td>
<td>0.2184</td>
<td>318</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (lvl 29)</td>
<td>0.1751</td>
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<tr>
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<td>Outrigger at top and mid-height</td>
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<tr>
<td>57-6</td>
<td>Outrigger at top, mid-height and 2/3 height</td>
<td>0.1863</td>
<td>0.2334</td>
<td>275</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>0.1797</td>
<td>0.2206</td>
<td>303</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>0.1812</td>
<td>0.2273</td>
<td>297</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>0.1830</td>
<td>0.2287</td>
<td>290</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>0.1905</td>
<td>0.2392</td>
<td>261</td>
</tr>
</tbody>
</table>
### Table 4.2

**Results for octagonal model**

<table>
<thead>
<tr>
<th>Octagonal Model</th>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency</th>
<th>Deflection at top</th>
<th>max. Storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mode 1 (Y-Dir)</td>
<td>Mode 2 (X-Dir)</td>
<td>DX</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
</tr>
</tbody>
</table>

#### 28-storeys

- **28-1** Without outrigger
  - Frequency: 0.415875 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.486169 Hz
    - Mode 2 (X-Dir): 0.486169 Hz
    - DX: 81 mm, DY: 110 mm
    - Storey drift: 1.14 mm, 1.73 mm

- **28-2** Outriggers at top
  - Frequency: 0.415875 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.486169 Hz
    - Mode 2 (X-Dir): 0.486169 Hz
    - DX: 81 mm, DY: 110 mm
    - Storey drift: 1.14 mm, 1.73 mm

- **28-3** Outrigger at mid height
  - Frequency: 0.415875 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.486169 Hz
    - Mode 2 (X-Dir): 0.486169 Hz
    - DX: 81 mm, DY: 110 mm
    - Storey drift: 1.14 mm, 1.73 mm

#### 42-storeys

- **42-1** Without outrigger
  - Frequency: 0.199592 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.268083 Hz
    - Mode 2 (X-Dir): 0.268083 Hz
    - DX: 309 mm, DY: 496 mm
    - Storey drift: 3.10 mm, 5.50 mm

- **42-2** Outrigger at top
  - Frequency: 0.236321 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.278908 Hz
    - Mode 2 (X-Dir): 0.278908 Hz
    - DX: 264 mm, DY: 328 mm
    - Storey drift: 2.70 mm, 3.70 mm

- **42-3** Outrigger at mid-height
  - Frequency: 0.254542 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.297667 Hz
    - Mode 2 (X-Dir): 0.297667 Hz
    - DX: 260 mm, DY: 305 mm
    - Storey drift: 2.80 mm, 3.50 mm

- **42-4** Outrigger at top and mid-height
  - Frequency: 0.284011 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.307887 Hz
    - Mode 2 (X-Dir): 0.307887 Hz
    - DX: 231 mm, DY: 227 mm
    - Storey drift: 2.70 mm, 2.40 mm

- **42-5** Double outrigger at top
  - Frequency: 0.251109 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.284212 Hz
    - Mode 2 (X-Dir): 0.284212 Hz
    - DX: 241 mm, DY: 278 mm
    - Storey drift: 2.50 mm, 3.20 mm

- **42-6** Double outrigger at mid-height
  - Frequency: 0.285162 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.313097 Hz
    - Mode 2 (X-Dir): 0.313097 Hz
    - DX: 233 mm, DY: 243 mm
    - Storey drift: 2.50 mm, 2.90 mm

#### 57-storeys

- **57-1** Without outrigger
  - Frequency: 0.16054 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.25698 Hz
    - Mode 2 (X-Dir): 0.25698 Hz
    - DX: 320 mm, DY: 632 mm
    - Storey drift: 2.50 mm, 5.40 mm

- **57-2** Outrigger at top
  - Frequency: 0.17992 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.26406 Hz
    - Mode 2 (X-Dir): 0.26406 Hz
    - DX: 290 mm, DY: 481 mm
    - Storey drift: 2.25 mm, 3.85 mm

- **57-3** Outrigger at 2/3 height (level 38)
  - Frequency: 0.19174 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.26836 Hz
    - Mode 2 (X-Dir): 0.26836 Hz
    - DX: 284 mm, DY: 437 mm
    - Storey drift: 2.30 mm, 3.50 mm

- **57-4** Outrigger at mid-height (level 29)
  - Frequency: 0.1867 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.26629 Hz
    - Mode 2 (X-Dir): 0.26629 Hz
    - DX: 293 mm, DY: 475 mm
    - Storey drift: 2.26 mm, 4.05 mm

- **57-5** Outrigger at top and mid-height
  - Frequency: 0.20284 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.27226 Hz
    - Mode 2 (X-Dir): 0.27226 Hz
    - DX: 268 mm, DY: 381 mm
    - Storey drift: 2.07 mm, 3.03 mm

- **57-6** Outrigger at top, mid-height and 2/3 height
  - Frequency: 0.22038 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.28032 Hz
    - Mode 2 (X-Dir): 0.28032 Hz
    - DX: 248 mm, DY: 316 mm
    - Storey drift: 2.05 mm, 2.35 mm

- **57-7** Double outrigger at top
  - Frequency: 0.18935 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.2697 Hz
    - Mode 2 (X-Dir): 0.2697 Hz
    - DX: 270 mm, DY: 419 mm
    - Storey drift: 2.11 mm, 3.41 mm

- **57-8** Double outrigger at mid-height
  - Frequency: 0.20499 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.20499 Hz
    - Mode 2 (X-Dir): 0.20499 Hz
    - DX: 272 mm, DY: 393 mm
    - Storey drift: 2.11 mm, 3.38 mm

- **57-9** Double outrigger at 2/3 height
  - Frequency: 0.21042 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.27729 Hz
    - Mode 2 (X-Dir): 0.27729 Hz
    - DX: 261 mm, DY: 356 mm
    - Storey drift: 2.13 mm, 2.69 mm

- **57-10** Double outrigger at top and mid-height
  - Frequency: 0.22654 Hz
  - Deflection at top:
    - Mode 1 (Y-Dir): 0.28414 Hz
    - Mode 2 (X-Dir): 0.28414 Hz
    - DX: 237 mm, DY: 295 mm
    - Storey drift: 1.85 mm, 2.34 mm
### Table 4.3

**Results for L-shaped model**

<table>
<thead>
<tr>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency (Hz)</th>
<th>Deflection at top</th>
<th>max. Storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mode 1 (Y-Dir)</td>
<td>Mode 2 (X-Dir)</td>
<td>DX</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.52037</td>
<td>0.562095</td>
<td>37</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
<td>0.541861</td>
<td>0.585756</td>
<td>33</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>0.549083</td>
<td>0.592318</td>
<td>33</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
<td>0.272314</td>
<td>0.293305</td>
<td>140</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
<td>0.288653</td>
<td>0.310919</td>
<td>120</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>0.293108</td>
<td>0.316163</td>
<td>120</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>0.306936</td>
<td>0.331709</td>
<td>106</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>0.300782</td>
<td>0.324645</td>
<td>106</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>0.314111</td>
<td>0.340426</td>
<td>104</td>
</tr>
<tr>
<td>57-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
<td>0.194008</td>
<td>0.208954</td>
<td>247</td>
</tr>
<tr>
<td>57-2</td>
<td>Outrigger at top</td>
<td>0.204443</td>
<td>0.218722</td>
<td>218</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (level 38)</td>
<td>0.20871</td>
<td>0.222793</td>
<td>215</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (level 29)</td>
<td>0.205941</td>
<td>0.220009</td>
<td>222</td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>0.215409</td>
<td>0.229007</td>
<td>199</td>
</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, mid-height and 2/3 height</td>
<td>0.226422</td>
<td>0.239875</td>
<td>180</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>0.212267</td>
<td>0.226722</td>
<td>197</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>0.216236</td>
<td>0.229961</td>
<td>204</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>0.220845</td>
<td>0.23482</td>
<td>192</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>0.231927</td>
<td>0.246057</td>
<td>168</td>
</tr>
</tbody>
</table>
4.10 COMPARISON AND DISCUSSION OF OUTPUT

4.10.1 Stiffness ratios

The values in are calculated according to the basic equation of linear stiffness for each model. Elastic linear stiffness is a characteristic of elastic modulus, area and length and is given in equation 4.9:

\[ k = \frac{AE}{H} \quad \text{N/m} \quad 4.9 \]

\( k \) = stiffness

\( A \) = area in m²

\( H \) = building height in m

\( E \) = elastic modulus in MPa

Ratio A and Ratio B are calculated by keeping \( E \) as constant, \( H \) is the total height of structure in meters and \( A \) is the plan area of layout in m², hence;

\[ k \propto \frac{A}{H} \]

Where; \( A = b \times d \), then

\[ k \propto \frac{b \times d}{H} \]

Thus, for each direction;

Ratio A ~ \( k \propto \frac{b}{H} \) and Ratio B ~ \( k \propto \frac{d}{H} \)

For Ratio C, the plan dimensions are replaced by the combined cross-sectional area of core walls and side walls (\( A_{\text{wall}} \)) in m² while \( H \) is replaced by the floor to floor height (\( H_{\text{floor}} \)) in metres (m). The value of elastic modulus is the same for all shear walls in all models. Therefore:

\[ k \propto \text{Cross-sectional area of core and side walls/floor to floor height} \]

\[ k \propto \frac{A_{\text{wall}}}{H_{\text{floor}}} \]
### Table 4.4

*Models’ stiffness ratios of plan dimensions to height*

<table>
<thead>
<tr>
<th>Model Type</th>
<th>b (m) (along X-axis)</th>
<th>Ratio A (b/H)</th>
<th>d (m) (along Y-axis)</th>
<th>Ratio B (d/H)</th>
<th>Ratio C ((A_{wall}/H_{floor}))</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>28-storey</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rectangular</td>
<td>80</td>
<td>0.82</td>
<td>30</td>
<td>0.31</td>
<td>11.20</td>
</tr>
<tr>
<td>Octagonal</td>
<td>60</td>
<td>0.61</td>
<td>60</td>
<td>0.61</td>
<td>10.14</td>
</tr>
<tr>
<td>L-shaped</td>
<td>80</td>
<td>0.82</td>
<td>60</td>
<td>0.61</td>
<td>21.30</td>
</tr>
<tr>
<td><strong>42-storey</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rectangular</td>
<td>80</td>
<td>0.54</td>
<td>30</td>
<td>0.20</td>
<td>14.36</td>
</tr>
<tr>
<td>Octagonal</td>
<td>60</td>
<td>0.41</td>
<td>60</td>
<td>0.41</td>
<td>16.05</td>
</tr>
<tr>
<td>L-shaped</td>
<td>80</td>
<td>0.54</td>
<td>60</td>
<td>0.41</td>
<td>25.53</td>
</tr>
<tr>
<td><strong>57-storey</strong></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Rectangular</td>
<td>80</td>
<td>0.40</td>
<td>30</td>
<td>0.15</td>
<td>36.76</td>
</tr>
<tr>
<td>Octagonal</td>
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<td>0.30</td>
<td>60</td>
<td>0.30</td>
<td>46.42</td>
</tr>
<tr>
<td>L-shaped</td>
<td>80</td>
<td>0.40</td>
<td>60</td>
<td>0.30</td>
<td>50.40</td>
</tr>
</tbody>
</table>

Table 4.4 provides comparison of each dimension of the plan, overall structural height and ratio of typical shear walls area to a typical floor height. The octagonal model has the lowest ratio, A in X-direction, which is perpendicular to along-wind actions or parallel to crosswind forces. The rectangular model has the lowest ratio, B in Y-direction, which is parallel to along-wind and perpendicular to crosswind actions. Ratio C is the lowest in the 28 storey octagonal model as well as in the 42 storey and 57 storey rectangular models.
4.10.2 Graphical representation of output

28 storey

**Figure 4.13.** Deflection comparison of 28 storey models

**Figure 4.14.** Fundamental frequency of 28 storey models

Deflections and frequencies of 28 storey models are compared in *Figure 4.13* and *Figure 4.14* respectively. The straight line of the octagonal model shows that insertions of outriggers have no effect on deflections. In the octagonal model, deflection in X-dir. is 80mm, while in Y-dir. It is 110 mm, because the core wall layout contribution of rigidity is higher in X-dir. than in Y-dir.

The graph is similar in the L-shaped model for frequency and deflection with higher values in Y-dir. The deflection values are almost unchanged at 28-2 and 28-3 in X-dir. and Y-dir. And frequency is also only slightly varied in X-axis and Y-axis.
Y-dir. in the rectangular model shows a trivial reverse curve in deflection. This stipulates that outriggers at the top provide better deflection control.

42 storey

Figure 4.15. Deflection comparison of 42 storey models.

Six variations of 42-storey models with various arrangements of belt-truss and outriggers are compared in

Figure 4.15 and Figure 4.16. In the octagonal model, the 42-4 curve is reversed and moved to the right. Although 42-2 and 42-5 have two outriggers levels, their arrangement affects deflection.

The rectangular model has reversed curvature, between 42-2 and 42-3, which shows that an outrigger at the top is more effective than in the middle. The values of deflection in X-dir. are very similar in the octagonal and rectangular model, whereas the L-shaped model has markedly less deflection. The X-dimension of the rectangular and L-shaped models is 80 m, but the shear wall contribution in the L-shaped model is higher than in the rectangular model.
The frequency variation in the octagonal model shows the outrigger affectivity. The 2\textsuperscript{nd} mode frequencies in all three plan options have a reverse curve at 42-5. Also, 42-4, 42-5 and 42-6 have double outriggers with different arrangements. The least effective is double outrigger at the top, i.e. 42-5. The provision of a mid-height outrigger has better effects due to the reversal of curvature at mid-height.

**57 storey model**

![Graph showing Fundamental Frequency of 42 storey models](image)

**Figure 4.16.** Fundamental Frequency of 42 storey models

![Graph showing Deflection comparison of 57 storey models](image)

**Figure 4.17.** Deflection comparison of 57 storey models

*Figure 4.17* shows the deflection curve for 10 models of 57 storeys. Generally, a sharp decline in deflection is observed as one outrigger level is inserted at the top.
floor. This trend continues up to 57-3 which has outriggers at 2/3 m height, however, the graph is reversed at 57-4 as the outrigger position has changed to mid-height of the model. Addition of an outrigger at two positions, i.e. at top and mid-height (57-5), again leads to decay of frequency.

The options 57-5, 57-7, 57-8 and 57-9 all have two outrigger levels but minimum deflection is achieved in both axes of 57-9, which has a double outrigger at 2/3 m height.

![Figure 4.18. Frequency comparison of 57 storey models](image)

The sharpest curve is for the octagonal model, as seen in Figure 4.18 and a milder curve is of rectangular plan. The marked increase of frequency by inserting three outrigger levels (57-6) and then an abrupt descent in values by providing a double outrigger at top (57-7) indicates that frequency is affected by the placement of bracings.
4.10.3 Comparison of output

4.10.3.1 Percentage deflection reductions

Percentage deflection reductions are calculated in equation 4.10 and the values of these reductions are listed in Table 4.5.

\[ \% \Delta_{\text{reduction}} = \frac{\Delta_{\text{model without outrigger}} - \Delta_{\text{any model arrangement}}}{\Delta_{\text{model without outrigger}}} \times 100 \quad 4.10 \]

Values in Table 4.5 show the maximum deflection decline obtained through various arrangements of trusses under wind action, in comparison with the model which is without belt-truss and outriggers.

The 28 storey octagonal model is not affected by any of the outrigger arrangements. The rectangular model has the least value at 28-2 while the L-shaped model has the lowest value at 28-3.

Rectangular and octagonal models have maximum reduction in deflection at 42-6 for X-axis and 42-4 for Y-axis. The L-shaped model has maximum deflection reduction at 42-6 in both axes.

In all 57 storey models, the maximum reduction of deflection is obtained at 57-10.
### Table 4.5

Percentage reduction in deflection

<table>
<thead>
<tr>
<th>Model title</th>
<th>Model arrangements</th>
<th>Rectangular</th>
<th>Octagonal</th>
<th>L-shaped</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>%ΔX</td>
<td>%ΔY</td>
<td>%ΔX</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
<td>11.50%</td>
<td>10.20%</td>
<td>0%</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>15.10%</td>
<td>8.10%</td>
<td>0%</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
<td>12.10%</td>
<td>14.60%</td>
<td>14.80%</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>14.90%</td>
<td>12.80%</td>
<td>16.10%</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>23.90%</td>
<td>23.40%</td>
<td>25.30%</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>20.20%</td>
<td>23.20%</td>
<td>22.30%</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>25.20%</td>
<td>22.70%</td>
<td>24.80%</td>
</tr>
<tr>
<td>57-storey</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>57-2</td>
<td>Outrigger at top</td>
<td>5.50%</td>
<td>5.20%</td>
<td>9.20%</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (level 38)</td>
<td>7.80%</td>
<td>5.40%</td>
<td>11.10%</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (level 29)</td>
<td>6.50%</td>
<td>3.80%</td>
<td>8.40%</td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>13.10%</td>
<td>11.00%</td>
<td>16.10%</td>
</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, mid-height and 2/3 height</td>
<td>20.10%</td>
<td>16.50%</td>
<td>22.40%</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>12.10%</td>
<td>11.40%</td>
<td>15.60%</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>13.80%</td>
<td>9.90%</td>
<td>15.00%</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>15.80%</td>
<td>12.40%</td>
<td>18.50%</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>24.30%</td>
<td>20.60%</td>
<td>25.80%</td>
</tr>
</tbody>
</table>
4.10.3.2 The frequency increments:

Frequency increments are calculated in equation 4.11 and given in Table 4.6. These show a very similar trend for all three model heights, and the highest frequency value is achieved with maximum outrigger and belt truss levels.

\[
\% f_{(increment)} = \frac{f_{(any\ model\ arrangement)} - f_{(model\ without\ outrigger)}}{f_{(model\ without\ outrigger)}} \times 100 \quad 4.11
\]

The frequency is a characteristic of stiffness; the more stiffness, the higher the frequency; however, frequency values also get affected by belt-truss and outrigger placement. For instance, 57-5, 57-7, 57-8 and 57-9 have two outrigger floors at various levels. Theoretically, the overall stiffness is same in these three options; however, the maximum percentage increment is attained at 57-9, which has a double outrigger level at 2/3rd height of the building. The placement of belt-truss and outriggers changes the centre of gravity of the model and impacts on the vertical curvature, which in turn affects the frequency of the model.

In the 42 storey model, comparison of 42-4, 42-5 and 42-6 shows that the maximum frequency increment is obtained at 42-6. Although theoretically the three of these have the same mass, changed truss placement changes the centre of gravity of the model and results in a changed frequency.
### Table 4.6
Percentage increment in frequency

<table>
<thead>
<tr>
<th>Model title</th>
<th>Model arrangements</th>
<th>Rectangular</th>
<th>Octagonal</th>
<th>L-shaped</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>%f₁st</td>
<td>%f₂nd</td>
<td>%f₁st</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0%</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
<td>5.00%</td>
<td>3.76%</td>
<td>0%</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>5.62%</td>
<td>7.17%</td>
<td>0%</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
<td>0%</td>
<td>0%</td>
<td>0%</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
<td>7.16%</td>
<td>5.00%</td>
<td>18.40%</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>7.51%</td>
<td>8.53%</td>
<td>27.53%</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>13.72%</td>
<td>13.05%</td>
<td>42.30%</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>12.45%</td>
<td>8.95%</td>
<td>25.81%</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>14.60%</td>
<td>15.99%</td>
<td>42.87%</td>
</tr>
<tr>
<td>57-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
<td>0.00%</td>
<td>0.00%</td>
<td>0.00%</td>
</tr>
<tr>
<td>57-2</td>
<td>Outrigger at top</td>
<td>3.77%</td>
<td>3.34%</td>
<td>12.07%</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (level 38)</td>
<td>4.69%</td>
<td>5.46%</td>
<td>19.44%</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (level 29)</td>
<td>4.08%</td>
<td>5.06%</td>
<td>16.30%</td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>7.36%</td>
<td>8.15%</td>
<td>26.35%</td>
</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, mid-height and 2/3 height</td>
<td>10.74%</td>
<td>12.68%</td>
<td>37.28%</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>6.84%</td>
<td>6.51%</td>
<td>17.95%</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>7.71%</td>
<td>9.75%</td>
<td>27.69%</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>8.78%</td>
<td>10.41%</td>
<td>31.07%</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>13.25%</td>
<td>15.50%</td>
<td>41.11%</td>
</tr>
</tbody>
</table>
4.11 CONCLUSION

This chapter has comprehensively presented the dynamic wind load calculations and applications to respective models. The results for deflections and frequencies were extracted and listed in the tables and represented graphically. These results illustrate the variation and trend of various belt-truss and outrigger combinations for different plan layouts and model heights, such as; the maximum percentage of frequency increment attained at 57-9; however, all four models, 57-5, 57-7 57-8 and 57-9 had two outrigger levels.

Wind action was responsive to the number and placement of belt-truss and outriggers. An increase in truss levels resulted in desirable serviceability conditions. The addition of truss levels, however, jeopardises the rentable space. A reduction in truss bracings was achieved by introducing more shear walls and by increasing the thicknesses of shear walls.

The findings of the above investigations are:

- 28-storey models return best results with the addition of single floor bracings at top level of models in three plan layouts.
- The single floor outrigger options in 42-storey models have varied outcomes in three plans layouts. However, the provision of a single floor truss at mid-height of the model has good deflection control.
- In double floor truss options, provision of truss at mid-height in 42-storey models provides the maximum reduction of lateral deflection.
- In the 57-storey model, if one floor of outrigger is required, then provision of one floor outrigger at 2/3rd height (measured from base) of model is the best option.
- In case of double belt-truss and outrigger floors in 57-storey model, the provision of secondary bracings at 2/3rd height (measured from ground level) of building is the most suitable alternative.
- Three single level bracings, one at the top of model, second at 2/3rd height from the base of model and third at mid-height of model, is more appropriate than providing two double outrigger floors at various locations along model height.
Chapter 5: Seismic Actions

5.1 INTRODUCTION

Earthquake loads are defined, calculated and applied in FEM (Strand7 R2.4.4, 2011) for seismic analysis. The seismic load calculations and application procedure are within the guidelines and scope of Australian seismic standard (AS 1170.4, 2007). Two types of earthquake loadings are adopted for seismic analysis and variables are calculated accordingly.

Results are listed at the end of chapter, and conclusions are drawn from the outcomes of the analysis.

5.2 EARTHQUAKE/SEISMIC ACTIONS

In structural engineering, seismic loading is the administration of earthquake-generated excitations to any structure or its components, in the form of forces and vibrations. These earthquake excitations are in horizontal and vertical directions. The vertical component of seismic action is usually counteracted by the weight of the structure, whereas horizontal actions are counteracted by the structural design.

Australia is in a moderate to low seismic zone, therefore it is usually sufficient in normal buildings to make lateral load resisting elements stronger and ensure the overall robustness of the structure against seismic action.

The terms “Earthquake” and “Seismic” will be used interchangeably; the word “standards” refers to relevant Australian standard and “loads”, “forces” and “action” means “applied forces” in this chapter.

5.3 LIMITATION AND SCOPE OF MODELLING

The focus of the thesis is on facilitating Australian engineers in their daily design task, as well as initiating a research process for the overall behaviour of a composite building structure under lateral loads. Therefore this chapter is formulated within the scope of Australian seismic standards (AS 1170.4, 2007).
5.4 PROCEDURE FOR CALCULATION OF EARTHQUAKE FORCES

The procedure for calculating the seismic action on a building is complicated and laborious; however, computer-aided three-dimensional analysis has made this arduous task relatively effortless and fast. Figure 5.1, taken from Earthquake standard (AS 1170.4, 2007), represents the procedure for the determination of earthquake actions on buildings within the frequency range of 0.2 Hz to 1.0Hz.

Figure 5.1. Seismic action design procedure

Figure 5.1 summarises the procedure of earthquake load generation. Adopted from Standard Australia (2007, p.17). Structural Design Actions Part 4: Earthquake actions in Australia (AS 1170.4-2007).

5.5 PARAMETERS FOR MODELS

Parameter selection is according to the recommendations and specifications of Australian standard (AS 1170.4, 2007). Assumptions are made where the required data is not available or outside the scope of this study. For example, the soil test data cannot be exact over the wide-ranging area extending between Bundaberg and Gladstone; for that reason, commonly occurring sub-soil class is assumed.
5.5.1 Selection of importance level for models

The models created for this study represent office buildings that come under “Class 5” of code (BCA, 2012). The office buildings usually house a great number of people, hence the risk to life and property could be significant; therefore, “ Importance Level 3” is chosen from standard (AS/NZS 1170.0, 2002).

5.5.2 Selection of site hazard factor for models

To maintain consistency in design, analysis and result comparison, the region (Region “C”) selected for wind analysis (chapter 4) is also used in the seismic analysis. The maximum value of seismic hazard is taken from table 3.2 of standard (AS 1170.4, 2007, p. 19) as “Z = 0.12” occurred between Gladstone and Bundaberg (Figure 5.2).

Figure 5.2. Queensland Earthquake Hazard map (partial) from AS 1170.4-2007

Figure 5.2 is seismic map of Queensland taken from Australian Standard; Structural Design Actions Part 4: Earthquake actions in Australia (AS 1170.4-2007, p. 25).
5.5.3 Selection of probability of occurrence for models

Standard (AS/NZS 1170.0, 2002, p. 26) recommends using a 25 year return period for a serviceability limit state. Therefore, the annual probability of exceedance (P) of 1/25 is selected with a probability factor ($k_p$) of 0.25 for the models (AS 1170.4, 2007, p. 18).

5.5.4 Selection of sub-soil class for models

Seismic loading is unique in that the medium (soil) which imposes the loading on a structure also provides it with support (Booth & Key 2006, 80).

The assessment of soil is outside the scope of this research, therefore a sub-soil classification that is most closely related and prevails in and around wind region “C” is assumed for calculation. Soil sub-class “$C_e$” provided in section 4 of standard (AS 1170.4, 2007) is selected for this thesis.

5.5.5 Selection of earthquake design category for models

Wilson et al. (2008, p. 26) state that standard (AS 1170.4, 2007) entails some sort of earthquake analysis for all types of buildings; they make use of a three-tiered approach, dependant on the Earthquake Design Category (EDC):

EDC I – Simple static check (10% weight of the structure)
EDC II – Static earthquake analysis
EDC III– Dynamic earthquake analysis
Table 2.1

<table>
<thead>
<tr>
<th>Importance level, type of structure (see Clause 2.2)</th>
<th>((k_pZ)) for site sub-soil class</th>
<th>Structure height, (h_s) (m)</th>
<th>Earthquake design category</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>E_s or D_s</td>
<td></td>
<td>Not required to be designed for earthquake actions</td>
</tr>
<tr>
<td>Domestic structure (housing)</td>
<td></td>
<td>Top of mof (\leq 8.5)</td>
<td>Refer to Appendix A</td>
</tr>
<tr>
<td>2</td>
<td>(\leq 0.05) (\leq 0.08)</td>
<td>(\leq 0.11) (\leq 0.14)</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>(&gt; 0.05) to (0.08) (&gt; 0.08) to (0.12)</td>
<td>(&gt; 0.11) to (0.17)</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>(&gt; 0.08) (&gt; 0.12)</td>
<td>(&gt; 0.17) (&gt; 0.21)</td>
<td>III</td>
</tr>
<tr>
<td>3</td>
<td>(\leq 0.08) (\leq 0.12)</td>
<td>(\leq 0.17) (\leq 0.21)</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>(&gt; 0.08) (&gt; 0.12)</td>
<td>(&gt; 0.17) (&gt; 0.21)</td>
<td>III</td>
</tr>
<tr>
<td>4</td>
<td>(\leq 0.12) (\leq 12)</td>
<td>(\geq 25) (\geq 50)</td>
<td>II</td>
</tr>
</tbody>
</table>

*Figure 5.3. Earthquake design category.*

*Figure 5.3* provides earthquake design categories to be used in seismic action estimations. Acquired from; Standard Australia (AS 1170.4, 2007, p. 16). Structural Design Actions Part 4: Earthquake actions in Australia.

EDC III category is chosen for models from Table 2.1 (*Figure 5.3*) of standard (AS 1170.4, 2007) for soil sub-class “C_e” and “\(k_pZ\)” values, established already for model/building heights. Standard (AS 1170.4, 2007) requires all EDC III structures to comply with clauses 5.2 and 5.5, and dynamic analysis must be carried out for such structures.

### 5.6 DYNAMIC ANALYSIS METHODS FOR MODELS

Standard (AS 1170.4, 2007) specifies dynamic analysis because it provides certain structural dynamic response characteristics that cannot be achieved with a static analysis technique.
5.6.1 Description of dynamic analysis

Wilson, Lam & Pham (2008, p. 27) explain that the three methods given for EDC I, EDC II and EDC III are all force-based methods.

Standard (AS 1170.4, 2007, p. 42) provides three types of analysis techniques for EDC III structures with a natural frequency between 1 Hz to 0.2 Hz. These are;

The horizontal response spectrum (HS)

The complex and random nature of ground motion renders it necessary to work with a general characterisation of ground motion. This is accomplished by using horizontal design response spectra to hypothesise the intensity and vibration content of ground motion at a given site (Bangash, 2011, p. 258).

This method is useful in model verification as well as providing an overall idea of building behaviour, storey forces, base shears, base overturning moments, displacements and storey drifts.

Site-specific design response spectra (SS)

SS is used to calculate dynamic loads for specific site sub-soil classifications and considers the soil profile and bed-rock ground motion. In this procedure, modal responses are combined in a statistical fashion to obtain the maximum value of building response.

Ground motion time histories

Time history analysis involves calculating the response of a structure at each increment of time when the base is subjected to specific ground motion (AS 1170.4, 2007, p. 42).

This method is not so popular, due to the fact that ground motion time histories usually are not available; the process of analysis is also complex and requires the designer to first perform HS and SS before subjecting the model to time-history loads.

5.6.2 Selection of seismic analysis methods for modelling

The retrieval of ground motion time histories are outside the scope of this study, hence this method is not used in the thesis. The models used in the thesis are analysed for HS and SS loadings.
5.7 APPLICATION OF SEISMIC ACTIONS ON MODEL

Based on the discussion in Section 5.6, the methods selected for dynamic earthquake analysis are:

- Horizontal design response spectrum (HS)
- Site-specific design response spectrum (SS)

The procedure for application of these loads in FEM (Strand7 R2.4.4, 2011) is described below, and detailed calculations are attached in Appendix C.

5.7.1 Horizontal design response spectrum (HS)

There are two ways of supplying the horizontal design response spectral loads in the programme (Strand7 R2.4.4, 2011):

- Manual in-put of forces on each storey in planer X-dir. and Y-dir.
- Auto-generation of the forces in X-dir. and Y-dir.

5.7.1.1 Manual force input

The most fundamental equation for calculating the base shear for horizontal design response spectrum loads is given in standard (AS 1170.4, 2007, p. 35) as shown in equation 5.1;

\[ V = \left[ k_p Z C_d (T) S_p / \mu \right] W_t \quad kN \tag{5.1} \]

Where:

- \( V \) = base shear
- \( T \) = period of vibration of structure being considered
- \( W_t \) =Seismic weight of structure taken as the sum of \( W_i \) (kN) for all levels.

In manual calculations, base shear is calculated by considering the first mode fundamental time period (\( T \)) attained from “Natural Frequency” analysis through Strand7 R2.4.4 (2011). The time period is then set against the selected sub-soil class in Table 6.4 of standard (AS 1170.4, 2007, p. 37), and spectral shape factor is evaluated using equation 5.2 and equation 5.3:
\[ C_d(T) = C(T) \frac{S_p}{\mu} \]  \hspace{1cm} \text{5.2}

\[ C_d(T) = k_p Z C_h(T) \frac{S_p}{\mu} \]  \hspace{1cm} \text{5.3}

Where:

- \( C_d(T) \) = horizontal design response spectrum as a function of period (\( T \))
- \( C(T) \) = elastic site hazard spectrum
- \( S_p \) = structural performance factor
- \( k_p \) = probability factor appropriate for the limit state under consideration
- \( Z \) = earthquake hazard factor
- \( \mu \) = structural ductility factor

Base shear calculated in equation 5.1 is distributed along the height (Figure 5.4) through equation 5.4 (AS 1170.4, 2007, p. 36). Because of \( k_{F,i} \), the top storey gets the maximum force, whereas the force value becomes zero as the lowest level is approached.

\[ F_i = k_{F,i} V \quad kN \]  \hspace{1cm} \text{5.4}

Where:

- \( k_{F,i} \) = seismic distribution factor for the \( i \)th level given in equation 5.5

\[ k_{F,i} = \frac{W_i h_i^k}{\sum_{j=1}^{n} (W_j h_j^k)} \]  \hspace{1cm} \text{5.5}

Where:

- \( h_i \) = of \( i \)th level above the base of structure in metres
- \( k \) = exponent depends on fundamental natural period of structure
- \( n \) = no of levels in structure
- \( W_j \) = seismic weight of structure or component at level \( j \) in kN
- \( W_i \) = seismic weight at \( i \)th level in kN given by equation 5.6 as;
\[ W_i = \sum G_i + \sum \psi_c Q_i \quad kN \quad \text{5.6} \]

Where:

- \( G_i \) = permanent action i.e. self weight or dead load
- \( \psi_c \) = imposed action (live load) combination factor (taken as 0.3)
- \( Q_i \) = imposed action (live load)

The forces obtained (Figure 5.4) are input in the model through an Excel sheet in X-dir. and Y-dir. (Figure 5.5), with due effects of torsion as given in standard (AS 1170.4, 2007, p. 40);

For each required direction of earthquake action, the earthquake actions shall be applied at the position calculated as \( \pm 0.1b \) from the nominal centre of mass, where \( b \) is the plan dimension of the structure at right angles to the direction of the action.

The standard (AS 1170.4, 2007, p. 40) further states: “This \( \pm 0.1b \) eccentricity shall be applied in the same direction at all levels and orientated to produce the most adverse torsion moment for the 100% and 30% loads”.

These forces are applied simultaneously in both directions, with 100% of the value in one direction and 30% of the value in the other direction (Table 5.1). The sample calculation is given in Figure 5.4, taken from Appendix C.
<table>
<thead>
<tr>
<th>Levels</th>
<th>Progressive Height</th>
<th>Total Structural Mass</th>
<th>Self weight of Structure</th>
<th>Slab Area</th>
<th>Super imposed load</th>
<th>Live Load</th>
<th>$W_{Fi}$</th>
<th>$W_{Di}$</th>
<th>$k_{Fi} - W_{Fi}$</th>
<th>$F_i = k_{Fi} F$</th>
</tr>
</thead>
<tbody>
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<td>15501</td>
<td>2950</td>
<td>4425</td>
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<td>22581</td>
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<td>0.0913</td>
<td>354</td>
</tr>
<tr>
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<td>15501</td>
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</tr>
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<td>15501</td>
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<td>8850</td>
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<tr>
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<td>4425</td>
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</tr>
<tr>
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<td>17378</td>
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<td>4425</td>
<td>8850</td>
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<tr>
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<td>4425</td>
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<td>0.0320</td>
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</tr>
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<td>5876190</td>
<td>0.0093</td>
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<td>0.0072</td>
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<td>17.5</td>
<td>18115.75</td>
<td>17765</td>
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<td>8850</td>
<td>24845</td>
<td>3327239</td>
<td>0.0053</td>
<td>20</td>
</tr>
<tr>
<td>4</td>
<td>14</td>
<td>18115.75</td>
<td>17765</td>
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<td>2217128</td>
<td>0.0036</td>
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<tr>
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<td>17765</td>
<td>2950</td>
<td>4425</td>
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<td>1388356</td>
<td>0.0022</td>
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<td>7</td>
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<td>24845</td>
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<td>17765</td>
<td>2950</td>
<td>4425</td>
<td>8850</td>
<td>24845</td>
<td>211908</td>
<td>0.0003</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 5.4. Abstract of excel sheet from distributed base shear table for each level (28 storey L-shaped).

Figure 5.5 Nodal force in Y-direction on a typical storey in L-shaped model

Load combinations with $+ve$ and $-ve$ signs are generated for the worst load case, giving the maximum deflections and worst storey drift (Table 3.2).
Table 5.1
Seismic load combination

<table>
<thead>
<tr>
<th>Combination</th>
<th>Comb 1</th>
<th>Comb 2</th>
<th>Comb 3</th>
<th>Comb 4</th>
<th>Comb 5</th>
<th>Comb 6</th>
<th>Comb 7</th>
<th>Comb 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear in X-dir. ($V_x$)</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>0.3</td>
<td>-0.3</td>
<td>0.3</td>
<td>-0.3</td>
</tr>
<tr>
<td>Shear in Y-dir. ($V_y$)</td>
<td>0.3</td>
<td>-0.3</td>
<td>0.3</td>
<td>-0.3</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
</tr>
</tbody>
</table>

5.7.1.2 Auto-generated seismic load

The alternative and fast method is “Auto-Seismic load generation” in Strand7 (Strand7 R2.4.4, 2011). This technique is very useful because:

- The time of entering forces on each level is saved;
- The re-checking of applied forces on each level is not required; hence the chances of errors are virtually eliminated;
- The non-structural mass can be provided at any eccentricity value.

For this method, non-structural mass is applied in the model as “plate non-structural mass (kg/m²)”, shown in Figure 5.6.

![Figure 5.6. Graphical representation of non-structural mass on typical L-shaped model](image)

In this study, two types of dead loads are used—structural self-weight and super-imposed dead loads. Strand7 R2.4.4 (2011) considers structural self-weight through element properties, which include member sizes, elastic modulus, densities, and poison ratios. Super-imposed dead weights account for ceiling, finishes,
partitions and service loads. Non-structural mass is a combination of superimposed dead weight and a factored live load given in equation 5.7.

Non structural mass = [Super imposed dead load + 0.3 Live load] kg 5.7

The non-structural mass was offset to account for the eccentricity in the models, as shown in Figure 5.7, according to standard (AS 1170.4, 2007, p. 43);

Where three-dimensional models are used for analysis, the effects of accidental torsion shall be accounted for, either by appropriate adjustments in the model, such as adjustment of mass locations, or by equivalent static procedures.

Figure 5.7. Auto-Seismic load case generation

The next step is to enter parameters (Figure 5.7) specific to Strand7 R2.4.4 (2011) HS loadings (Strand7 Webnotes, 2011):

Structural mass ($\alpha$) – the mass amplification factor, taken as “1”.

Non-Structural mass ($\phi$) – the factor which could be the reciprocal of imposed action combination factor ($\varphi_c$), taken as “1”.

Base shear factor ($\beta$) – Strand7 R2.4.4 (2011) seismic factor is calculated as in Strand7 Web notes (2011), given in equation 5.8. The term $C_h(T_1)$ is taken for selected site sub-soil class from standard (AS 1170.4, 2007) shown in figure 5.8.
\[ \beta = \left( k_p Z C_h(T_1) S_p / \mu \right) \]

5.8

**Elevation exponent (k)** – this was same as given in equation 5.5.

**Gravity Direction** – X-axis and Y-axis are taken as planner directions in the models and Z-axis is the vertical axis along the building height. Hence, the direction of acceleration due to gravity would be along the Z-axis.

**Excitation direction** – \( V_x \) and \( V_y \) are in the X-axis and Y-axis respectively. The value is “1.00” for the direction the load is being considered.

**Base Elevation** – this defines the base elevation of the model. The models are drawn from base to the top end, therefore the elevation from base is taken as “0.0”.

**Add non-structural mass from**..... – If there is more than one non-structural load case then the designer has the option to choose from them.

Models are solved by “Linear Static solver” using above parameters under load combinations in Table 3.2 for the worst seismic load effects.

<table>
<thead>
<tr>
<th>Period (seconds)</th>
<th>( A_r ) Rock</th>
<th>( B_r ) Rock</th>
<th>( C_r ) Shallow soil</th>
<th>( D_r ) Deep or soft soil</th>
<th>( E_r ) Very soft soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>2.35 (0.8)*</td>
<td>2.94 (1.0)*</td>
<td>3.68 (1.3)*</td>
<td>3.68 (1.1)*</td>
<td>3.68 (1.1)*</td>
</tr>
<tr>
<td>0.1</td>
<td>2.35</td>
<td>2.94</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>0.2</td>
<td>2.35</td>
<td>2.94</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>0.3</td>
<td>2.35</td>
<td>2.94</td>
<td>3.68</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>0.4</td>
<td>1.76</td>
<td>2.20</td>
<td>3.12</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>0.5</td>
<td>1.41</td>
<td>1.76</td>
<td>2.90</td>
<td>3.68</td>
<td>3.68</td>
</tr>
<tr>
<td>0.6</td>
<td>1.17</td>
<td>1.47</td>
<td>2.08</td>
<td>3.30</td>
<td>3.68</td>
</tr>
<tr>
<td>0.7</td>
<td>1.01</td>
<td>1.26</td>
<td>1.79</td>
<td>2.83</td>
<td>3.68</td>
</tr>
<tr>
<td>0.8</td>
<td>0.88</td>
<td>1.10</td>
<td>1.56</td>
<td>2.48</td>
<td>3.68</td>
</tr>
<tr>
<td>0.9</td>
<td>0.78</td>
<td>0.98</td>
<td>1.39</td>
<td>2.20</td>
<td>3.42</td>
</tr>
<tr>
<td>1.0</td>
<td>0.70</td>
<td>0.88</td>
<td>1.25</td>
<td>1.98</td>
<td>3.08</td>
</tr>
<tr>
<td>1.2</td>
<td>0.59</td>
<td>0.73</td>
<td>1.04</td>
<td>1.65</td>
<td>2.57</td>
</tr>
<tr>
<td>1.5</td>
<td>0.47</td>
<td>0.59</td>
<td>0.83</td>
<td>1.32</td>
<td>2.05</td>
</tr>
<tr>
<td>1.7</td>
<td>0.37</td>
<td>0.46</td>
<td>0.65</td>
<td>1.03</td>
<td>1.60</td>
</tr>
<tr>
<td>2.0</td>
<td>0.26</td>
<td>0.33</td>
<td>0.47</td>
<td>0.74</td>
<td>1.16</td>
</tr>
<tr>
<td>2.5</td>
<td>0.17</td>
<td>0.21</td>
<td>0.30</td>
<td>0.48</td>
<td>0.74</td>
</tr>
<tr>
<td>3.0</td>
<td>0.12</td>
<td>0.15</td>
<td>0.21</td>
<td>0.33</td>
<td>0.51</td>
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<tr>
<td>3.5</td>
<td>0.086</td>
<td>0.11</td>
<td>0.15</td>
<td>0.24</td>
<td>0.38</td>
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<tr>
<td>4.0</td>
<td>0.066</td>
<td>0.083</td>
<td>0.12</td>
<td>0.19</td>
<td>0.29</td>
</tr>
<tr>
<td>4.5</td>
<td>0.052</td>
<td>0.065</td>
<td>0.093</td>
<td>0.15</td>
<td>0.23</td>
</tr>
<tr>
<td>5.0</td>
<td>0.042</td>
<td>0.053</td>
<td>0.075</td>
<td>0.12</td>
<td>0.18</td>
</tr>
</tbody>
</table>

Equations for spectra:

\[ 0 < T \leq 0.1 \]
\[ 0.1 < T \leq 1.5 \]
\[ T > 1.5 \]

Figure 5.8. Soil-sub class in AS 1170.4, 2007.

Figure 5.8 provides soil subclass to be used in seismic action estimations. Acquired from; Standard Australia (AS 1170.4, 2007, p. 16). Structural Design Actions Part 4: Earthquake actions in Australia.
5.7.1.3 **Comparison and agreement of manual and auto-generated loads.**

- The values of base shear and base overturning moment obtained from the above two procedures are compared and examined, to check if these are in agreement, in order to proceed further.
- Once satisfied with the output then the rest of the models with various outrigger arrangements are analysed with “Seismic load case (i.e. auto-generated loads)” option of Strand7 R2.4.4, (2011).

5.7.2 **Site-specific response spectra**

SS is the commonly used dynamic analysis method based on sub-soil class and natural frequency analysis requiring two solvers in Strand7 R2.4.4 (2011), i.e. natural frequency solver and response spectrum solver:

**Natural Frequency solver**—standard (AS 1170.4, 2007, p. 43) recommends 90% mass participation, but not categorically, in three-dimensional models:

In three-dimensional analysis, where structures are modelled so that modes are not those of the seismic-force-resisting system, then all modes not part of the seismic-force-resisting system shall be ignored. Further, all modes with periods less than 5% of the fundamental natural period of the structure (<0.05\(T_1\)) may be ignored.

Generally in models, the values of period reduce more than 5% of the fundamental period beyond the range of the first ten modes. Hence, models are solved for the first 10 modes, and the frequencies of these modes are included only.

**Spectral response solver:** this solver is based on the spectral shape factor \([C_h(T)]\), which is provided for various sub-soil classes and time period ranges in standard (AS 1170.4, 2007), as given by equation 5.9;

\[
C_d(T) = \left[ k_p Z C_h(T) S_p / \mu \right] 5.9
\]
In equation 5.9, the term \( C_d(T) \) is set by the normalised response spectrum entered in the form of a table in FEM (Strand7 R2.4.4, 2011), shown in Figure 5.9. The remaining terms are also calculated and entered for the direction vector (Figure 5.10) in X-dir. and Y-dir.

The frequency file was included from the frequency analysis and only converged modes were used.
Two types of methods of calculating the maximum structural response are available in Strand7 (Strand7 R2.4.4, 2011)—SRSS (Square root of sum of square) and CQC (complete quadratic combination). Since the comparison of various methods is not a requirement of this thesis, the method which is commonly used (in practice), SRSS, is selected for analysis.

5.8 MODEL OUTPUT

5.8.1 Horizontal design response spectrum (HS)

Analysis results for HS loads include frequency, deflection at the top floor and maximum storey drift for the most critical load combination. For example, combination 2 (Table 3.2) is most critical in X-dir. of the rectangular model (Table 5.2), whereas combination 5 (Table 3.2) is critical in the L-shaped model (Table 5.4).

Rectangular models (HS)

28-3 has maximum deflection in the Y-axis (Table 5.2). 42-5 has minimum deflection in the Y-axis. 42-6 has the maximum deflection although it has double outriggers at mid height. Storey drift is also maximum in 42-6.

Table 5.2 shows the maximum deflection at 57-1 and minimum at 57-7, with a percentage difference of only 6% in the Y-axis. The provision of a double truss floor in 57-7 only gave a 1.042% deflection reduction compared to a single floor truss (57-2). 57-5 and 57-6 have the same deflection values, although 57-5 has two levels of outrigger and 57-6 has three levels of truss.
### Table 5.2

Results for rectangular models (HS)

<table>
<thead>
<tr>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency (Y-Dir)</th>
<th>Frequency (X-Dir)</th>
<th>Deflection at top Comb 2</th>
<th>Deflection at top Comb 6</th>
<th>max. Storey drift Comb 2</th>
<th>max. Storey drift Comb 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Critical load combination</td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>Mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td><strong>28- storeys</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.34581</td>
<td>0.37295</td>
<td>120.5</td>
<td>151.7</td>
<td>5.0</td>
<td>6.0</td>
</tr>
<tr>
<td>28-2</td>
<td>Outriggers at top</td>
<td>0.36407</td>
<td>0.38782</td>
<td>119.1</td>
<td>148.7</td>
<td>5.1</td>
<td>6.7</td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>0.36587</td>
<td>0.40004</td>
<td>116.8</td>
<td>153.3</td>
<td>5.2</td>
<td>6.2</td>
</tr>
<tr>
<td><strong>42-storeys</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
<td>0.20946</td>
<td>0.25332</td>
<td>106.7</td>
<td>166.7</td>
<td>3.50</td>
<td>5.70</td>
</tr>
<tr>
<td>42-2</td>
<td>Outrigger at top</td>
<td>0.22153</td>
<td>0.26273</td>
<td>106</td>
<td>161.6</td>
<td>3.40</td>
<td>5.90</td>
</tr>
<tr>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>0.22091</td>
<td>0.2685</td>
<td>105.2</td>
<td>166.4</td>
<td>3.50</td>
<td>5.80</td>
</tr>
<tr>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>0.23164</td>
<td>0.27734</td>
<td>105.9</td>
<td>164.9</td>
<td>3.50</td>
<td>6.10</td>
</tr>
<tr>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>0.23072</td>
<td>0.2706</td>
<td>105.7</td>
<td>160.4</td>
<td>3.50</td>
<td>5.70</td>
</tr>
<tr>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>0.23099</td>
<td>0.28213</td>
<td>106</td>
<td>169.6</td>
<td>3.60</td>
<td>6.00</td>
</tr>
<tr>
<td><strong>57-storeys</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1</td>
<td>Without outrigger</td>
<td>0.19521</td>
<td>0.23844</td>
<td>133</td>
<td>205</td>
<td>4.00</td>
<td>6.00</td>
</tr>
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<td>57-2</td>
<td>Outrigger at top</td>
<td>0.20042</td>
<td>0.24352</td>
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<td>192</td>
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<td>5.00</td>
</tr>
<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (lvl* 38)</td>
<td>0.20132</td>
<td>0.24766</td>
<td>124</td>
<td>196</td>
<td>4.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (lvl* 29)</td>
<td>0.19979</td>
<td>0.24562</td>
<td>126</td>
<td>197</td>
<td>4.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>0.20453</td>
<td>0.2505</td>
<td>124</td>
<td>194</td>
<td>4.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, 1/2 height &amp; 2/3 height</td>
<td>0.20917</td>
<td>0.25871</td>
<td>120</td>
<td>194</td>
<td>3.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>0.20471</td>
<td>0.24828</td>
<td>122</td>
<td>190</td>
<td>4.00</td>
<td>5.00</td>
</tr>
<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>0.20388</td>
<td>0.25254</td>
<td>124</td>
<td>198</td>
<td>4.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>0.20692</td>
<td>0.25641</td>
<td>121</td>
<td>196</td>
<td>4.00</td>
<td>6.00</td>
</tr>
<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>0.21217</td>
<td>0.26188</td>
<td>118</td>
<td>192</td>
<td>3.00</td>
<td>6.00</td>
</tr>
</tbody>
</table>

* lvl = level
**Octagonal models (HS)**

The deflection of the 28-storey model shows no impact of the outrigger although the storey drift shows change with the introduction of outriggers.

In the 42-storey model, deflection in the Y-direction is higher with one outrigger at mid-height, but less with a double outrigger at the top. 42-1 has a minimum deflection in the X-axis, whereas 42-5 has a minimum deflection in the Y-axis.

There is maximum deflection in 57-1 (Table 5.3) without any truss and with the addition of a double outrigger at the top, the deflection drops down to minimum value, with a huge difference of about 63%. However, minimum deflection cannot be achieved in models with three outrigger levels and even with four outrigger levels, as in 57-10. The top truss is more effective in reversing the deflection curvature. Minimum storey drift is achieved in 57-6 and 57-9.
Table 5.3
Results for octagonal models (HS)

<table>
<thead>
<tr>
<th>Octagonal models</th>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency Mode 1 (Y-Dir)</th>
<th>Frequency Mode 2 (X-Dir)</th>
<th>Deflection at top Comb 1 DX mm</th>
<th>Deflection at top Comb 5 DY mm</th>
<th>max. Storey drift Comb 1 DX mm</th>
<th>max. Storey drift Comb 5 DY mm</th>
</tr>
</thead>
<tbody>
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<td>Critical load combination</td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
<td>mm</td>
<td></td>
<td></td>
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<td>28- storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.33521</td>
<td>0.37841</td>
<td>104.7</td>
<td>149.4</td>
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<td>Outriggers at top</td>
<td>0.33521</td>
<td>0.37841</td>
<td>104.7</td>
<td>149.4</td>
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<td>6.8</td>
<td></td>
</tr>
<tr>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>0.33521</td>
<td>0.37841</td>
<td>104.7</td>
<td>149.4</td>
<td>3.8</td>
<td>6.8</td>
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<tr>
<td>42- storeys</td>
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<td></td>
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<td>Without outrigger</td>
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<td>0.27891</td>
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<td>0.29767</td>
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<td>42-4</td>
<td>Outrigger at top and mid-height</td>
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<td>0.28421</td>
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<td>99.0</td>
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<td>0.28876</td>
<td>0.31403</td>
<td>121.0</td>
<td>155.0</td>
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<td>57- storeys</td>
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<td></td>
</tr>
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<td>57-1</td>
<td>Without outrigger</td>
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<td>57-2</td>
<td>Outrigger at top</td>
<td>0.19661</td>
<td>0.29208</td>
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<td>205.9</td>
<td>3.61</td>
<td>7.21</td>
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<tr>
<td>57-3</td>
<td>Outrigger at 2/3 height (lvl 38)</td>
<td>0.20700</td>
<td>0.29725</td>
<td>211.6</td>
<td>105.1</td>
<td>4.09</td>
<td>8.18</td>
<td></td>
</tr>
<tr>
<td>57-4</td>
<td>Outrigger at mid-height (lvl 29)</td>
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<td>0.29211</td>
<td>101.8</td>
<td>224.1</td>
<td>4.01</td>
<td>9.29</td>
<td></td>
</tr>
<tr>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>0.21161</td>
<td>0.29719</td>
<td>106.8</td>
<td>205.2</td>
<td>3.91</td>
<td>7.22</td>
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</tr>
<tr>
<td>57-6</td>
<td>Outrigger at top, 1/2 height &amp; 2/3 height</td>
<td>0.22910</td>
<td>0.30577</td>
<td>118.5</td>
<td>204.2</td>
<td>4.45</td>
<td>2.61</td>
<td></td>
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<tr>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>0.20425</td>
<td>0.29625</td>
<td>192.6</td>
<td>96.8</td>
<td>3.38</td>
<td>6.45</td>
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<tr>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>0.20793</td>
<td>0.29661</td>
<td>107.7</td>
<td>222</td>
<td>4.16</td>
<td>9.18</td>
<td></td>
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<td>Double outrigger at 2/3 height</td>
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<td>0.30552</td>
<td>114.5</td>
<td>209</td>
<td>4.47</td>
<td>2.81</td>
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<tr>
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<td>Double outrigger at top and mid-height</td>
<td>0.22975</td>
<td>0.30556</td>
<td>117</td>
<td>201.9</td>
<td>4.18</td>
<td>6.96</td>
<td></td>
</tr>
</tbody>
</table>

* lvl = level
**L-shaped models (HS)**

The minimum displacement in the 28-storey model is with no outriggers (Table 5.4), while when a truss is placed at mid-height, the displacement is maximum.

The 42-storey model’s deflection recedes with the addition of an outrigger (Table 5.4). 42-5 has minimum deflection in the Y-axis, that is, 30% reduction. 42-6 has minimum deflection in the X-axis, that is, 13.5% reduction. The storey drift does not have significant difference in their values.

The L-shaped model (Table 5.4) has a higher deflection in the Y-axis relative to the X-axis. The maximum reduction is around 23% in Y-dir. and 19%in X-dir. In the X-axis, 57-4 has a higher deflection than 57-1. 57-3 and 57-9 have a maximum storey drift in the Y-axis; both of them have outriggers placed at 2/3rd height.
Table 5.4
Results for L-shaped models (HS)

### L-shaped models

<table>
<thead>
<tr>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency</th>
<th>Deflection at top</th>
<th>max. Storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mode 1 (Y-Dir)</td>
<td>Mode 2 (X-Dir)</td>
<td>DX Comb 1</td>
</tr>
<tr>
<td>Critical load combination</td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1 Without outrigger</td>
<td>0.52037</td>
<td>0.56209</td>
<td>124.7</td>
<td>138.8</td>
</tr>
<tr>
<td>28-2 Outriggers at top</td>
<td>0.44586</td>
<td>0.47974</td>
<td>128.1</td>
<td>152.1</td>
</tr>
<tr>
<td>28-3 Outrigger at mid height</td>
<td>0.45011</td>
<td>0.48377</td>
<td>133.1</td>
<td>157.9</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>42-1 Without outrigger</td>
<td>0.22487</td>
<td>0.24065</td>
<td>160.5</td>
<td>186.9</td>
</tr>
<tr>
<td>42-2 Outrigger at top</td>
<td>0.23804</td>
<td>0.25554</td>
<td>143.2</td>
<td>171.8</td>
</tr>
<tr>
<td>42-3 Outrigger at mid-height</td>
<td>0.24107</td>
<td>0.25946</td>
<td>149</td>
<td>179</td>
</tr>
<tr>
<td>42-4 Outrigger at top and mid-height</td>
<td>0.25232</td>
<td>0.27295</td>
<td>163.9</td>
<td>132.8</td>
</tr>
<tr>
<td>42-5 Double outrigger at top</td>
<td>0.24803</td>
<td>0.26734</td>
<td>160.2</td>
<td>130.9</td>
</tr>
<tr>
<td>42-6 Double outrigger at mid-height</td>
<td>0.25759</td>
<td>0.2799</td>
<td>138.8</td>
<td>172.2</td>
</tr>
<tr>
<td>57-storeys</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>57-1 Without outrigger</td>
<td>0.19017</td>
<td>0.20406</td>
<td>171.6</td>
<td>197.2</td>
</tr>
<tr>
<td>57-2 Outrigger at top</td>
<td>0.19761</td>
<td>0.21153</td>
<td>154.1</td>
<td>180.1</td>
</tr>
<tr>
<td>57-3 Outrigger at 2/3 height (lvl 38)</td>
<td>0.20093</td>
<td>0.21518</td>
<td>153.3</td>
<td>180.3</td>
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<tr>
<td>57-4 Outrigger at mid-height (lvl 29)</td>
<td>0.19926</td>
<td>0.21348</td>
<td>182.8</td>
<td>156.2</td>
</tr>
<tr>
<td>57-5 Outrigger at top and mid-height</td>
<td>0.20584</td>
<td>0.22019</td>
<td>180.4</td>
<td>152</td>
</tr>
<tr>
<td>57-6 Outrigger at top, 1/2 height &amp; 2/3 height</td>
<td>0.21384</td>
<td>0.22871</td>
<td>151.3</td>
<td>182.2</td>
</tr>
<tr>
<td>57-7 Double outrigger at top</td>
<td>0.2034</td>
<td>0.21746</td>
<td>148.1</td>
<td>175</td>
</tr>
<tr>
<td>57-8 Double outrigger at mid-height</td>
<td>0.20723</td>
<td>0.22206</td>
<td>156.7</td>
<td>186</td>
</tr>
<tr>
<td>57-9 Double outrigger at 2/3 height</td>
<td>0.21066</td>
<td>0.22499</td>
<td>153.2</td>
<td>183.4</td>
</tr>
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<td>57-10 Double outrigger at top and mid-height</td>
<td>0.21817</td>
<td>0.23378</td>
<td>149.1</td>
<td>181</td>
</tr>
</tbody>
</table>

* lvl = level
5.8.2 Site-specific design response spectra (SS)

The outputs of SS analysis are extracted for the worst load combinations in the X-axis and Y-axis.

**Rectangular model (SS)**

The minimum deflection and storey drift is obtained in 28-2 (Table 5.5) in the X-axis and Y-axis.

19% reduction in the Y-axis and 16% reduction in the X-axis are achieved with the introduction of a double outrigger at the top level of 42 storeys (42-5). This model also has minimum storey drift in Y-dir.

The 57 storey model has higher return of deflection when subjected to SS (Table 5.5). 14% and 16% drops in deflection are achieved with two double levels of outriggers in 57-10, in Y-dir. and X-dir. respectively. A 6% drop is noted in 57-2 and a 10% drop is attained in 57-7, with a single and double floor outrigger at the top in the Y-axis respectively.
### Table 5.5

**Results for rectangular models (SS)**

<table>
<thead>
<tr>
<th>Critical load combination</th>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency Mode 1 (Y-Dir.)</th>
<th>Frequency Mode 2 (X-Dir.)</th>
<th>Deflection at top DX Comb 2 mm</th>
<th>Deflection at top DY Comb 6 mm</th>
<th>max. Storey drift DX Comb 2</th>
<th>max. Storey drift DY Comb 6</th>
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<tbody>
<tr>
<td>28-storeys</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28-1</td>
<td>Without outrigger</td>
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<td>0.34581</td>
<td>0.37295</td>
<td>71.6</td>
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<td>Outriggers at top</td>
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<td>88.48</td>
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</tr>
<tr>
<td>42-1</td>
<td>Without outrigger</td>
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<td>251.8</td>
<td>420.7</td>
<td>6.3</td>
<td>10.9</td>
</tr>
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<td>0.25871</td>
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<td>408.1</td>
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<tr>
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<td>0.25254</td>
<td>255.4</td>
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<td>0.25641</td>
<td>255.4</td>
<td>432</td>
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<tr>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
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<td>0.21217</td>
<td>0.26188</td>
<td>232.9</td>
<td>395.3</td>
<td>5.9</td>
<td>10</td>
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* lvl = level
Octagonal models (SS)

The 28-storey octagonal model in HS and SS has no impact of outriggers (Table 5.3 & Table 5.6).

Decrements of 42% and 18% are obtained in the Y-axis and X-axis of 42-2 respectively (Table 5.6). 42-5 shows a frequency drop, however, the total numbers of outriggers are same in 42-4 and 42-5.

57-10 shows 39% and 19% deflection decline in Y and X axes respectively. However, 38% and 18% drops are seen in 57-6 with three outrigger level in Y-axis and X-axis. The addition of a fourth outrigger only contributes to 1% extra reduction.

Table 5.6
Results for octagonal models (SS)

<table>
<thead>
<tr>
<th>Octagonal models</th>
<th>Critical load combination</th>
<th>Model title</th>
<th>Model arrangements</th>
<th>Frequency Mode 1 (Y-Dir)</th>
<th>Frequency Mode 2 (X-Dir)</th>
<th>Deflection at top Comb 1</th>
<th>Deflection at top Comb 5</th>
<th>max. Storey drift Comb 1</th>
<th>max. Storey drift Comb 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>28-storeys</td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28-1</td>
<td>Without outrigger</td>
<td>0.33521</td>
<td>0.37841</td>
<td>130.46</td>
<td>181.05</td>
<td>1.35</td>
<td>2.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28-2</td>
<td>Outriggers at top</td>
<td>0.33521</td>
<td>0.37841</td>
<td>130.46</td>
<td>181.05</td>
<td>1.35</td>
<td>2.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>28-3</td>
<td>Outrigger at mid height</td>
<td>0.33521</td>
<td>0.37841</td>
<td>130.46</td>
<td>181.05</td>
<td>1.35</td>
<td>2.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-storeys</td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-1</td>
<td>Without outrigger</td>
<td>0.18252</td>
<td>0.22026</td>
<td>347.6</td>
<td>616.2</td>
<td>3.9</td>
<td>5.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-2</td>
<td>Outrigger at top</td>
<td>0.21203</td>
<td>0.23267</td>
<td>325.3</td>
<td>475.1</td>
<td>3.8</td>
<td>3.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-3</td>
<td>Outrigger at mid-height</td>
<td>0.22282</td>
<td>0.25285</td>
<td>320.1</td>
<td>455.8</td>
<td>4</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-4</td>
<td>Outrigger at top and mid-height</td>
<td>0.24723</td>
<td>0.26514</td>
<td>283.7</td>
<td>358.8</td>
<td>3.9</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-5</td>
<td>Double outrigger at top</td>
<td>0.22280</td>
<td>0.24042</td>
<td>303.7</td>
<td>396.3</td>
<td>3.8</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>42-6</td>
<td>Double outrigger at mid-height</td>
<td>0.24550</td>
<td>0.27093</td>
<td>286.8</td>
<td>394</td>
<td>3.5</td>
<td>3.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-storeys</td>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-1</td>
<td>Without outrigger</td>
<td>0.17979</td>
<td>0.28636</td>
<td>311.01</td>
<td>633.88</td>
<td>4.6</td>
<td>16.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-2</td>
<td>Outrigger at top</td>
<td>0.19571</td>
<td>0.29195</td>
<td>286.06</td>
<td>518.75</td>
<td>8.5</td>
<td>13.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-3</td>
<td>Outrigger at 2/3 height (lvl 38)</td>
<td>0.20665</td>
<td>0.29734</td>
<td>281.55</td>
<td>487.03</td>
<td>4.07</td>
<td>11.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-4</td>
<td>Outrigger at mid-height (lvl 29)</td>
<td>0.19603</td>
<td>0.29199</td>
<td>298.51</td>
<td>547.53</td>
<td>4.31</td>
<td>14.53</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-5</td>
<td>Outrigger at top and mid-height</td>
<td>0.21074</td>
<td>0.29707</td>
<td>276.08</td>
<td>459.66</td>
<td>8.25</td>
<td>11.74</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-6</td>
<td>Outrigger at top, 1/2 height &amp; 2/3 height</td>
<td>0.22813</td>
<td>0.30565</td>
<td>253.65</td>
<td>394.77</td>
<td>7.82</td>
<td>9.93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-7</td>
<td>Double outrigger at top</td>
<td>0.20330</td>
<td>0.29611</td>
<td>271.1</td>
<td>471.35</td>
<td>8.27</td>
<td>11.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-8</td>
<td>Double outrigger at mid-height</td>
<td>0.20719</td>
<td>0.29649</td>
<td>289.73</td>
<td>499.38</td>
<td>4.15</td>
<td>13.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-9</td>
<td>Double outrigger at 2/3 height</td>
<td>0.22280</td>
<td>0.30575</td>
<td>260.35</td>
<td>421.54</td>
<td>4.1</td>
<td>10.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>57-10</td>
<td>Double outrigger at top and mid-height</td>
<td>0.22884</td>
<td>0.30543</td>
<td>252.25</td>
<td>388.59</td>
<td>3.63</td>
<td>9.98</td>
</tr>
</tbody>
</table>

* lvl = level
**L-shaped models (SS)**

The lowest deflections and storey drifts are obtained in 28-2 SS (Table 5.7).

42-4 has the least deflections in both directions; 42-5 has the least deflection in Y-dir.; and 42-6 has the least deflection of 21.8% in the X-axis (Table 5.7).

Overall, SS analysis provides lower deflection (Table 5.7) values compared with HS analysis (Table 5.4), and 57-7 seems to be the best option with the minimum values.

Table 5.7

*Results for L-shaped models (SS)*

<table>
<thead>
<tr>
<th>Critical load combination</th>
<th>Frequency</th>
<th>Deflection at top</th>
<th>max. Storey drift</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mode 1 (Y-Dir)</td>
<td>Mode 2 (X-Dir)</td>
<td>DX</td>
</tr>
<tr>
<td></td>
<td>Hz</td>
<td>Hz</td>
<td>Comb 1</td>
</tr>
<tr>
<td>28-storeys</td>
<td></td>
<td></td>
<td>Hz</td>
</tr>
<tr>
<td>28-1 Without outrigger</td>
<td>0.42814</td>
<td>0.45847</td>
<td>72.37</td>
</tr>
<tr>
<td>28-2 Outriggers at top</td>
<td>0.44586</td>
<td>0.47974</td>
<td>61.27</td>
</tr>
<tr>
<td>28-3 Outrigger at mid height</td>
<td>0.45316</td>
<td>0.48761</td>
<td>59.52</td>
</tr>
<tr>
<td>42-storeys</td>
<td></td>
<td></td>
<td>Hz</td>
</tr>
<tr>
<td>42-1 Without outrigger</td>
<td>0.22487</td>
<td>0.24065</td>
<td>344.14</td>
</tr>
<tr>
<td>42-2 Outrigger at top</td>
<td>0.23804</td>
<td>0.25553</td>
<td>301.07</td>
</tr>
<tr>
<td>42-3 Outrigger at mid-height</td>
<td>0.24107</td>
<td>0.25946</td>
<td>301.32</td>
</tr>
<tr>
<td>42-4 Outrigger at top and mid-height</td>
<td>0.25232</td>
<td>0.27295</td>
<td>268.75</td>
</tr>
<tr>
<td>42-5 Double outrigger at top</td>
<td>0.24803</td>
<td>0.26734</td>
<td>272.34</td>
</tr>
<tr>
<td>42-6 Double outrigger at mid-height</td>
<td>0.25533</td>
<td>0.27712</td>
<td>269.07</td>
</tr>
<tr>
<td>57-storeys</td>
<td></td>
<td></td>
<td>Hz</td>
</tr>
<tr>
<td>57-1 Without outrigger</td>
<td>0.19017</td>
<td>0.20406</td>
<td>112.9</td>
</tr>
<tr>
<td>57-2 Outrigger at top</td>
<td>0.19761</td>
<td>0.21153</td>
<td>110.5</td>
</tr>
<tr>
<td>57-3 Outrigger at 2/3 height (lvl* 38)</td>
<td>0.20093</td>
<td>0.21518</td>
<td>114.4</td>
</tr>
<tr>
<td>57-4 Outrigger at mid-height (lvl* 29)</td>
<td>0.19926</td>
<td>0.21348</td>
<td>114.9</td>
</tr>
<tr>
<td>57-5 Outrigger at top and mid-height</td>
<td>0.20584</td>
<td>0.22019</td>
<td>112.1</td>
</tr>
<tr>
<td>57-6 Outrigger at top, mid-height &amp; 2/3 height</td>
<td>0.21384</td>
<td>0.22871</td>
<td>114.3</td>
</tr>
<tr>
<td>57-7 Double outrigger at top</td>
<td>0.20340</td>
<td>0.21746</td>
<td>109.6</td>
</tr>
<tr>
<td>57-8 Double outrigger at mid-height</td>
<td>0.20723</td>
<td>0.22206</td>
<td>116.2</td>
</tr>
<tr>
<td>57-9 Double outrigger at 2/3 height</td>
<td>0.21012</td>
<td>0.22502</td>
<td>115.6</td>
</tr>
<tr>
<td>57-10 Double outrigger at top and mid-height</td>
<td>0.21817</td>
<td>0.23378</td>
<td>113.9</td>
</tr>
</tbody>
</table>

* lvl = level
5.9 OUTPUT RESULTS IN GRAPHICAL FORM

The following graphs are developed based on the values given in Table 5.2, Table 5.3 & Table 5.4 to graphically compare results of various options.

5.9.1 Horizontal Design response spectrum (HS)

Figure 5.11 shows the 28-storey deflection under HS. Apart from a higher decrement in deflection in the Y-axis, the trend of the L-shaped graph is similar in both axes. However, the rectangular model shows two different types of curve in two axes. The top outrigger has the best impact on the rectangular shape.

Figure 5.12 shows an unsymmetrical trend in Figure 5.12. The rectangular shape demonstrates less fluctuation in X-dir. than in Y-dir. The graph of the rectangular
model is a straight line in X-dir. 42-1 has minimum deflection in the octagonal model and maximum deflection in the L-shaped model in the X-axis.

\[\text{Figure 5.13. Deflection comparison of 57 storey model (HS)}\]

The rectangular shape is quite undisturbed, while the octagonal graph has several curvatures, which means that it is affected by various outrigger options. The L-shaped model shows intermediate behaviour (\textit{Figure 5.13}). The graph of the octagonal shaped model is opposite in both axes; for instance, 57-3 has the least deflection value in Y-axes and maximum value in X-axes. A double outrigger at the top of the octagonal shaped model shows the smallest amount of deflection in Y-dir., and the second highest value in X-dir.
5.9.2 Site-specific design response spectra (SS)

Under HS load (Figure 5.14), the octagonal model has not responded to any outrigger placement and the graph is a straight line. The rectangular model shows a similar curve in the X-axis and Y-axis, that is, the top outrigger has the lowest deflection value. The L-shaped model is the only one with a varied graph in both axes.

![Deflection comparison of 28 storey models (SS)](image)

Figure 5.14. Deflection comparison of 28 storey models (SS)

All three models show almost the same curvature, with a varied magnitude of deflection (Figure 5.15). The curvature of the rectangular graph is similar in both axes under SS loading, unlike those under HS loadings. The L-shaped model also has
an analogous curve in both axes, responding with almost the same values in 42-4, 42-5 and 42-6 storey options. The octagonal model also follows the same path with almost the same curve in both axes, and a difference in magnitude of values.

![Figure 5.16. Deflection comparison of 57-storey models (SS)](image)

Unlike the HS loading, the SS loading of the 57-storey options are fairly regular (Figure 5.16). For instance, the rectangular and L-shape models have higher curvature and the difference in values is also less drastic. The outriggers do not have much impact in changing the deflection values in these two. However, the octagonal model shows slightly more impact with the introduction of truss levels along its height.
5.10 CONCLUSION

The results of HS and SS show similarity in certain models and are completely different in others. The deflection values of SS are higher in tall models, while in the 28-storey models, HS gives larger values.

It is also noted that increased self-weight due to the insertion of outriggers actually increases deflections because increased self-weight results in increased base shear and consequently increased horizontal force.

It is extrapolated from the overall results that the best possible location of belt-truss and outriggers is at the top of the structure. Truss placed at mid-height and at 2/3rd height (measured from base) of model has less impact on horizontal deflection and is of secondary significance. Hence; mainly lateral deflection control is attributed to the top level truss.

The outcomes presented in this chapter are summarised as follows:

*Using the horizontal design response spectrum, results achieved are:*

- Under horizontal design response spectrum (HS) loading, the provision of a truss gives no advantage in the 28-storey model.
- With both single and double floor truss options, placement of truss at the top level gives the best results in 42-storey rectangular and L-shaped plan.
- In the octagonal plan, the provision of truss results in an increase of deflection in 42-storey model. Hence; it can be ascertained that the octagonal model behaves best without any belt-truss and outriggers under HS loading.
- Among single floor belt-truss and outriggers options, provision of truss at 2/3rd height (measured from base) is the best choice in 57-storey rectangular and L-shaped prototypes.
- In double floor outrigger options, a top level double floor truss is the best alternative among 57-storey rectangular and L-shaped models.
- For the octagonal plan, the results are varied and the best option is the provision of a single floor truss at top level of 57-storey.
Conclusions drawn from use of site-specific design response spectra are:

- Provision of belt-truss and outrigger at top floor of the 28-storey model reduces deflection most efficiently.

- If a single floor truss is required in 42-storey building, the best option in rectangular and L-shaped plans is the provision of a single floor truss at the top level of the model. However, a single floor truss at mid-height is the best option in the octagonal plan model.

- In the case of double truss floors in the 42-storey model, their provision at mid-height is the best option in the octagonal model, while truss placement at the top level serves best in the rectangular and L-shaped models.

- Maximum deflection reduction is obtained with two single floor outriggers placed at top level and mid-height of the 42-storey model in all three plan layouts.

- In the 57-storey model, comparison of single floor truss options showed that it works best when given at the top level in the rectangular and L-shaped models.

- A single floor truss has best results when given at 2/3rd height from the base of the octagonal 57-storey model.

- A double floor truss behaves the same way as a single floor truss in 57-storey models that is best deflection control is achieved with top level truss. However, due to more mass and more stiffness in the double floor truss option, higher deflection reduction is obtained.

- If more than three levels of truss are required in 57-storey buildings rather than double floors truss, staggered truss floors provide effective deflection reduction and frequency control.
Chapter 6: **Results and Conclusion**

6.1 **INTRODUCTION**

This chapter summarises the results and outcomes presented in the previous chapters of this thesis, outlining the choice of best options, and proposing analytical models for deflection prediction on the basis of the research findings.

The final part of this chapter suggests directions for further research and investigation.

The words “belt truss and outrigger”, “outrigger”, “bracing” or “truss” will be used interchangeably in the following paragraphs, and have the same meaning. The words “lateral displacement”, “deflection”, “horizontal deflection”, “horizontal displacement” will be used interchangeably and have same meaning.

6.2 **SUMMARY OF RESULTS;**

The outcomes are established after careful examination of results from Strand7 (R2.4.4, 2011) wind and seismic analysis in chapters 4 and 5 respectively. The summarised results incorporate wind and seismic loadings and are valid for stiffness ratios given in Table 4.4. The following results highlight the options best suited to buildings of various heights for deflection control:

- Single floor belt-truss and outriggers at the top level of buildings is the best option for deflection reduction under wind and seismic loadings for buildings up to 100 m height.
- For buildings with heights between 100 m to 150 m, single floor belt-truss and outrigger at the top level is best suited for rectangular and L-shaped layouts under seismic loading.
- Under wind loads, single floor outrigger level at mid-height gives the best lateral displacement control in multi-storey buildings with height ranges from 100 m to 150 m in rectangular and L-shaped plans.
- For wind and seismic actions, in the single floor belt-truss and outrigger options, provision of bracings at mid-height of a building best suit plans of equal horizontal dimensions within 100 m to 150 m height range.
Among single floor bracing options, the provision of belt-truss and outriggers placed at 2/3\textsuperscript{rd} height (measured from base) of buildings is the best alternative in rectangular and L-shaped buildings within a height range of 150 m to 200 m under wind actions.

If seismic loading is critical and double floor bracings are required, the addition of double floor belt-truss and outriggers at top level of building in rectangular and L-shaped models is the best alternative for buildings with elevations of 150 m to 200 m.

For multi-storey buildings with elevations from 150 m to 200 m, the provision of belt-truss and outriggers at 2/3\textsuperscript{rd} height (measured from base) give better deflection control in rectangular and L-shaped models if subject to wind loads.

For models with equal plan (i.e. horizontal) dimensions and vertical height ranges from 150 m to 200 m, among single floor belt-truss and outrigger options, placement of secondary bracings at the top level returns the best results under wind and seismic actions.

Staggered levels of outriggers, i.e. two or three single truss floors at various levels such as mid-height and 2/3\textsuperscript{rd} height from base of building, yield better results than double truss levels in buildings with heights ranging from 150 m to 200 m.

6.3 BEST OPTION SELECTION

6.3.1 Basis of option selection

The assessment of various models is based on the frequency, storey drift and deflection results.

Australian Standards (AS 1170.4 : 2007, AS/NZS 1170.2 : 2011) limit the frequency range of structures to accommodate calculations within Australian Standards’ capacity. Most models in this study have a frequency within 0.2 Hz to 2.0 Hz (Australian Standard limits).

AS/NZS 1170.0 (2002) suggests lateral deflection limits under wind and seismic actions. Deflection in models is highly variable and affected by the number
and placement of belt-truss and outrigger along model height. Hence, this is the deciding criterion of best option choice.

Australian Standard (AS 1170.4, 2007) provides procedure for determination of storey drift; however, it does not specify upper and lower limits. The maximum storey drifts in X-dir. and Y-dir. are 8 mm and 16 mm respectively, obtained in the octagonal model under SS loadings (Table 5.6). Storey drifts do not show significant variations under different combinations of belt-truss and outrigger, hence are not considered as a deciding factor in best option selection.

Therefore, models with high frequency and least lateral deflections are deemed to be desirable. Hence, a comparison is compiled drawing on data in Table 4.1, Table 4.2, Table 4.3, Table 5.2, Table 5.3, Table 5.4, Table 5.5, Table 5.6 & Table 5.7 for best possible options for various lateral loadings.

6.3.2 Best model option

The maximum height of each model (section 3.4, chapter 3) was limited to comply with Australian Standards (i.e. AS 1170.4, 2007, AS/NZS 1170.2, 2011). Best model options are selected with respect to the number and placement of belt-truss and outriggers and horizontal forces acting on prototypes.

Recapitulating, the maximum height model, 57-storeys, is 199.5 metres (given the storey heights as 3.5m), which is the maximum allowable height as per Australian Standards (AS/NZS 1170.2, 2011). This is chosen to study the effects of wind and earthquake loadings according to Australian Standards on the maximum given building elevation.

42-storey (147.0 m high) is the most common type of multi-storey rise within the Australian urban environment. Many offices and residential buildings are constructed around this height.

28-storey is half the height of the 57-storey model at 98 m. This height was selected to establish a comparison and to find out the benefits of outriggers on such a low elevation.

This study is a comparative study of:

- Three heights of models.
- Three plan layouts.
• Placements of belt-truss and outrigger at various heights in the models.

• Number of belt-truss and outrigger levels provided in the models.

Following are the best options among the models analysed in chapter 4 and chapter 5 with respect to wind and seismic loadings.

**57-storey**

Wind – 57-10 double belt-truss and outrigger at top level and mid-height

HS – 57-7 double floor belt-truss and outrigger at top level

SS – 57-10 double floor belt-truss and outrigger at top level and mid-height

**42-storey**

Wind – 42-4; single floor belt-truss and outrigger at top level and mid-height

HS – 42-5; double floor belt-truss and outrigger at top level

SS – 42-5; double floor belt-truss and outrigger at top level

**28-storey**

28-2 single floor belt-truss and outrigger for all (lateral) loading.

### 6.4 SELECTION OF PROTOTYPE FOR ANALYTICAL MODEL

The model most responsive to the combination of belt-truss and outrigger and showing distinct behaviour when subjected to horizontal forces was selected for the analytical model.

57-storey (i.e. 199.5 m high) models (as discussed in chapters 4 and chapter 5) are highly receptive to the addition and placement of belt-truss and outrigger. Belt-truss and outrigger are secondary bracings which act with primary bracings (i.e. RCC core wall/shear wall) and help to create stabilisation against horizontal actions in buildings.

One of the main aims of this study is to establish an analytical model based on finite element analysis of composite high-rise buildings subjected to horizontal
loadings. Hence, three heights and three layouts are investigated with various arrangements of secondary bracings, as described in the preceding chapters. With careful examination, it is ascertained that the 57-storey model is the most suitable option and fulfils the criteria of high-rise buildings, hence is the focus of this analytical comparison. The other two prototypes (with 98 m and 147 m height) are very useful for comparison and contrasting of results. However, the addition of secondary bracing is usually more effective for high-rise buildings (i.e. buildings with height more than 150 m).

The tallest height that can be undertaken within the scope of Australian standards (AS 1170.4, 2007 & AS/NZS 1170.2, 2011) is 200 m. Therefore, an office building prototype with 3.5 m floor-to-floor height can rise up to 199.5 m.

6.5 ANALYTICAL MODEL BASED ON MAXIMUM HEIGHT PROTOTYPE

In the succeeding paragraphs, examination of 57-storey models has been carried out with respect to;

- Height of models.
- Plan layout of models.
- Placement and number of belt truss and outriggers along the model height.

6.5.1 Examination of maximum height models for analytical comparison

Three types of comparison are taken into consideration for the analytical proposal:

Option a. *Single outriggers:* 57-2, 57-3 and 57-4 are compared.

Option b. *Double outriggers:* 57-7, 57-8 and 57-9 are compared.

Option c. *Number of outrigger levels:* 57-2, 57-5 and 57-6 are compared.

The results in chapter 4 (Table 4.1, Table 4.2 & Table 4.3) and chapter 5 (Table 5.2, Table 5.3, Table 5.4, Table 5.5, Table 5.6 & Table 5.7) were evaluated and conclusions were drawn.

Various graphs are plotted (given in appendix D) in order to get a certain forecast for deflections for other similar options. The graphs showed curves with
reverse and non-reverse curvature, whereas some are simply straight lines due to no change in deflection values. Those graphs showing reverse curvature or a straight line are not considered, because they could not be interpolated into the linear equation. Hence graphs with non-reverse curves are considered and transformed into linear equations.

6.5.2 Rationalisation

These formulae are proposed for building heights between 150 m to 200 m, based on stiffness as summarised in Table 4.6, chapter 4. Stiffness is a characteristic of elastic modulus, area and length given in equation 6.1, reproduced from equation 4.10, chapter 4;

\[ k = \frac{AE}{H} \quad \frac{N}{m} \quad 6.1 \]

k = stiffness

\( A = \) area in ‘m\(^2\’

\( H = \) model height in ‘m’

\( E = \) elastic modulus in ‘MPa’

Ratio A and Ratio B are derived from equation 6.1 as;

\[ k \propto \frac{A}{H} \quad \text{keeping E constant} \]

Where:

\( A = b \times d \)

\( b = \) plan width in ‘m’

\( d = \) plan depth/length in ‘m’

Then:

\[ k \propto \frac{(b \times d)}{H} \]

Thus, for each direction:

Ratio A \sim k \propto \frac{b}{H} \quad \text{and} \quad \text{Ratio B} \sim k \propto \frac{d}{H} \]

Ratio C is given as:

\[ k \propto \text{Cross-sectional area of core and side walls / Floor to floor height} \]
\[ k \propto \frac{A_{\text{wall}}}{H_{\text{floor}}} \]

Table 6.1 is reproduced from Table 4.4 chapter 4.

Table 6.1

*Plan dimension to height ratios*

<table>
<thead>
<tr>
<th>Model Type</th>
<th>(b) (m)</th>
<th>Ratio A ((b/H))</th>
<th>(d) (m)</th>
<th>Ratio B ((d/H))</th>
<th>Ratio C ((A_{\text{wall}}/H_{\text{floor}}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rectangular</td>
<td>80</td>
<td>0.40</td>
<td>30</td>
<td>0.15</td>
<td>37</td>
</tr>
<tr>
<td>Octagonal</td>
<td>60</td>
<td>0.30</td>
<td>60</td>
<td>0.30</td>
<td>46</td>
</tr>
<tr>
<td>L-shaped</td>
<td>80</td>
<td>0.40</td>
<td>60</td>
<td>0.30</td>
<td>50</td>
</tr>
</tbody>
</table>

6.5.3 Proposal for analytical model

The results presented in chapter 4 and chapter 5 are closely examined and conclusions are proposed in the form of formulae. These, however, are the findings of preliminary research. Further research and investigation is required before these can become part of a structural design calculation process. After careful assessment, formulae for prediction of lateral deflection are proposed. The applicability and effectiveness of following proposals are applicable to:

- Building heights within 150 m to 200 m.
- The stiffness outlined in Table 6.1.
- Models analysed under Australian Standards recommended procedures.
**Proposal - 1**

This formula is proposed to predict maximum lateral displacement in buildings subjected to wind or seismic (SS) loadings for the three layouts used in this study. This formula is based on the use of single floor belt-truss and outriggers along the building height. This formula is valid for:

- Building structures satisfying section 6.5.3 of thesis.
- Single floor belt-truss and outrigger placed along building height.
- Model subjected to one type of lateral loads at a time.
- Number of belt-truss and outrigger levels < 20 (i.e. $n < 20$).

The proposed formula is given in Equation 6.2:

\[
\Delta_{\text{def}} = \frac{(20 - n)}{0.0375} \text{ mm} \quad 6.2
\]

**Proposal - 2**

The formula is proposed for the prediction of lateral deflection for buildings subjected to seismic (HS) loading for all three layouts used in this study. This is based on the placement of double floor belt-truss and outrigger levels along the model height. The formula is applicable for:

- Building structures satisfying section 6.5.3 of thesis.
- Double floor belt-truss and outrigger placed along building height.
- Vertical distance ($h_{\text{truss}}$) of double floor belt-truss and outrigger measured from ground level in metres.

This is given in Equation 6.3 as:

\[
\Delta_{\text{def}} = \frac{(510 - h_{\text{truss}})}{2} \text{ mm} \quad 6.3
\]
6.6 FUTURE RESEARCH PROSPECTS

This research and investigation has opened a vast scenario for future studies. The proposed formulae can be used in future research and applied to models for further study.

In this subject, extensive explorations can be conducted with a wide range of variables and characteristics. Further studies may be carried out by adding the following variations to the research prototypes;

- Varied stiffness on varied building levels.
- Introduction of a soft storey in model.
- Different loadings on different levels of buildings.
- Placement of outrigger and belt truss on 1/4th, 3/4th, 4/5th, 3/5th etc. building height. These can be placed individually or in combination;
- Providing outriggers in one direction and maintaining the stability by RCC core in the other direction.


