WIND LOAD EFFECTS ON HIGH-RISE BUILDING

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A project report submitted in partial fulfilment of the requirements for the award of Bachelor of Engineering (Honours) Civil Engineering

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DECLARATION

I hereby declare that this project report is based on my original work except for citations and quotations which have been duly acknowledged. I also declare that it has not been previously and concurrently submitted for any other degree or award at UTAR or other institutions.

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APPROVAL FOR SUBMISSION

I certify that this project report entitled **"WIND LOAD EFFECTS ON HIGH RISE BUILDING"** was prepared by **TAN YEE JIN** has met the required standard for submission in partial fulfilment of the requirements for the award of Bachelor of Engineering (Honours) Civil Engineering at Universiti Tunku Abdul Rahman.

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ABSTRACT

Wind-related disasters such as windstorms, hurricanes, and sandstorms are destructive to local citizens. Numerous tragedies caused by the failure of structural members had been recorded in the past decades. Hence, structural and wind engineer plays a vital role in performing a constructive wind load analysis for tall buildings. This study aimed to evaluate the effects of wind load on high-rise buildings. The objectives of this study are to evaluate the parameters used in the wind load calculation, to investigate the effects of different wind loading evaluation and to compare the building response under the ultimate condition for member forces in the shear wall and support reaction. The evaluation of wind load parameters was accomplished according to European Standard, British Standard, and Australia/New Zealand Standard. In this study, linear analysis for two building models (i.e., single building model and twin building model with a podium) were carried out using SCIA Engineer based on Eurocode (EN 1991-1-4:2004). The output of support reactions on shear walls and columns for both building models were evaluated. Under ultimate limit state condition, the percentage difference between 14% to 57% is obtained by comparing single building model and twin building with podium Other than that, under wind load, a percentage difference of between 0% to 20% is obtained. Hence, the difference in reactions between the two building models is verified. The factors that contribute to the percentage difference are identified. Wind tunnel test and CFD simulation are recommended to improve the accuracy of results and provide firm justifications.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS / ABBREVIATIONS

A_{ref}	reference area of structure or structural component, m^2
A_z	area of loading, m^2
C_a	size effect factor
C_d	drag coefficients
C_{dyn}	dynamic response factor
C_{fig}	aerodynamic shape factor
C_o	orography factor
C_p	net pressure coefficient
$C_{pe,1}$	external pressure coefficient at $1m^2$
$C_{pe,10}$	external pressure coefficient at $1m^2$
C_{pe}	external pressure coefficient
C_{pi}	internal pressure coefficient
C_r	dynamic augmentation factor
C_r	wind speed factor of roughness factor
F_w	wind force
$I_v(z)$	turbulence intensity
K_d	directionally factor
K_l	local pressure factors
M_h	shape multiplier
M_d	wind directional multiplier
M_s	shielding multiplier
M_t	topographic multiplier
$M_{z,cat}$	terrain/height multiplier
S_h	topographic increment
S_a	altitude factor
S_b	terrain & building factor
S_c	fetch factor
S_d	direction factor
S_p	probability factor
S_s	seasonal factor

S_t	turbulence factor
T_c	fetch adjustment factor
T_{out}	temperature
T_t	turbulence adjustment factor
$V_{des,0}$	building orthogonal wind speed, m/s
$V_{sit,\beta}$	site wind speed, m/s
c_d	dynamic coefficient
c_{dir}	directional factor
$c_e(z)$	exposure factor
c_f	force coefficient of structure or structural component
$c_o(z)$	roughness coefficient
c_{pe}	pressure coefficient of external surface
$c_r(z)$	orography coefficient
$c_s c_d$	structural factor
c_{season}	seasonal factor
f_{tk}	maximum tensile strength, MPa
f_{yk}	yield strength, MPa
g_k	characteristic permanent action
g_t	gust peak factor
p_i	internal wind pressure, Pa
p_z	design wind pressure at height z, Pa
q_b	basic wind pressure, Pa
q_e	dynamic pressure (effective wind speed), Pa
q_k	characteristic variable action
q_p	extreme wind pressure, Pa
$q_p(z_e)$	peak velocity pressure, Pa
q_s	dynamic pressure, Pa
v_R	regional 3s gust wind speed, m/s
$v_{b,0}$	initial wind speed, m/s
v_b	basic wind speed, m/s
v_e	effective wind speed, m/s
v_m	average wind speed, m/s
$v_m(z)$	mean wind speed, m/s

w_e	external wind pressure, Pa
z_e	reference height of external pressure, m
ρ_{air}	density of air which taken as $1.2 kg/m^3$
A	area of loading, m^2
B	crosswind width of structure, m
d	alongwind width, m
h	height, m
Hz	frequency
p	design wind pressure, Pa
s	shielding parameter
Δ	lateral displacement between floors, mm
E	elastic modulus, MPa
G	shear modulus, MPa
p	net wind pressure, Pa

AS/NZS	Australia/New Zealand Standard
ASCE	American Society of Civil Engineers
BLWTL	boundary layer wind tunnel laboratory test
BS	British Standard
BSI	British Standard Institution
CEN	European Committee for Standardization
CFD	computational fluid dynamics
COL	column
CPE	coefficient of external pressure
CPI	coefficient of internal pressure
EC	European Standard
GIS	Geographic Information System
IF	interference factor
MRI	mean incurrence interval
MS	Malaysia Standard
SW	shear wall
ULS	ultimate limit state

LIST OF APPENDICES

CHAPTER 1

INTRODUCTION

1.1 General Introduction

Urbanisation and growth of economies worldwide have contributed to a significant impact on urban planning of cities in the construction industry. The construction of tall buildings has becoming a trend due to its economic measure in providing more space with higher occupancy. According to Hallebrand and Jakobsson (2016), tall buildings are buildings that consist of a large number of floors high slenderness ratio compared to typical buildings. Thus, lateral load must be considered as a significant contribution to design loads for tall buildings. Although there is no specifications made to define tall buildings, most of the standard codes have assessed the building's height as the principal factor in the categorisation of parameters for computation of wind response (Holmes, 2001).

Contemplation of structural integrity of tall buildings is necessary to withstand undeniable natural forces of gravity, wind and earthquakes. Therefore, the wind force is necessary to be examined and considered. Wind load design is compulsory for tall buildings due to its increasing effect following the increment of the height of buildings. Either alongwind surface or crosswind surface of buildings takes into consideration of wind load design due to the phenomenon of vortex-shedding during high wind velocity (Mendis, et al., 2007). Besides, tall buildings also take into account the torsional building response.

Conservative wind load design of tall buildings is compulsory to mitigate or minimise vibration and horizontal deflection caused by wind actions. Wind-related disasters such as windstorms, hurricanes, tornadoes and sandstorms, have also been recorded worldwide. Similarly, in Malaysia, numerous tragedies have been ascertained. The aftermath of wind-related disasters such as roofs being torn off, buildings being struck by debris and falling of structural elements are commonly recognised.

Undeniably, the wind effect on tall buildings is influenced by several parameters despite the height of buildings. According to Awida (2010), numerous researches on structural analysis and experiments such as Wind Tunnel test was carried out to examine the importance of the parameters for wind response. In term of physical characteristics of buildings, factors such as aerodynamic modifications, shape and aspect ratio of buildings are determined. Besides, evaluation of architectural mitigations on physical aspects of buildings has also been implemented. Parameters such as interference effect, topographic effect, terrain condition, wind directionality, seasonal variation and gust factor act as external factors for buildings response. These parameters which alter the wind effects on buildings are also considered in standard codes for wind load design.

Different standard codes of design are referred to as wind load evaluation is executed in various countries. In Malaysia, MS 1553:2002 is applied to implement wind load analysis according to a localised region. The important parameters such as basic wind speed, site exposure multipliers, dynamic response factor, external and internet pressure coefficient could be obtained. Hence, it is crucial to perform comparison of standard codes in order to examine the appropriateness and effective wind load design. In order to accomplish a comparative analysis of wind codes, the application of engineering software is considered due to its economical and efficient way of performing tasks. In this study, SCIA Engineer is utilised to carry out wind load analysis compliance with various standard codes.

1.2 Problem Statement

In recent decades, numerous cases of damages of buildings and losses of lives caused by wind disaster had been recorded in Malaysia. It is identified that frequent occurrence of thunderstorms is recorded during the transition period of monsoon in which higher wind speed was detected. Damages of buildings and roofs being torn off have been recorded on the west coast of Peninsular Malaysia and Borneo states of Sabah and Sarawak. Besides, Macalister Road tragedy was the recognised accident that happened in Penang Island, which caused by the falling of the structural component of the tall building. Negligence of design wind speed by engineers for the structural component was the reason which had led to the tragedy.

The researches or analysis about the topics of comparison and evaluation of wind load parameters and wind load calculations in various codes were in deficiency to be studied and investigated. In particular, distinguish of wind load analysis implemented within European Standard, British Standard, and Australia/New Zealand Standard were lacking. The previous practice of wind load design on a high-rise building in Malaysia is referred to MS1533 in which some parameters are not considered. For example, structural factor and size factor which are considered in Eurocode (EN 1991-1-4:2004) are neglected. This occurrence might lead to consideration of smaller wind pressure and load than the actual condition which tends to cause accidents due to wind disaster

1.3 Aim and Objectives

The aim of this study is to evaluate the wind load effects on high-rise buildings response. In order to achieve the designated aim, the following objectives have been determined:

- To evaluate parameters used in wind load calculation according to European Standard, British Standard, and Australia/New Zealand Standard.
- To investigate the differences of wind loading evaluation according to European Standard, British Standard, and Australia/New Zealand Standard.
- To compare the building response under ultimate condition for member forces in the shear wall and support reaction.

1.4 Scope and Limitations of the Study

This study consists of the comparative study of wind load on tall buildings based on three codes, which are European Standard (EN 1991-1-4:2004), British Standard (BS 6399-2:1997), and Australia/New Zealand Standard (AS/NZS 1170.2:2011). To carry out the wind load evaluation, SCIA Engineer is adopted.

There are two buildings modelled for the comparative study of the project. Two models are created with span to depth ratio (h/d) of 1:5 and the aspect ratio (h/b) of 1:2.5. The first model is a rectangular building, which comprises eight floors with a floor height of 3 m and a total height of 24 m. Besides, basic horizontal structural components such as beams and slabs, while vertical components like columns and shear walls are included in the model. Shear walls are located at the centre of the building model as inner vertical components.

For the second model, twin rectangular building sitting on a podium is modelled. Same as the first model, a rectangular building comprises the exact number of floors, floor height is proposed. Besides, an identical span and aspect ratio are also decided for the second model. Basic horizontal structural components such as beams and slabs, columns and shear walls, and they are allocated in the same way as in the first model. Moreover, the podium of the second model with a dimension of 32000 mm x 8000 mm is decided to have five floors and floor height of 3 m.

In this project, both building models developed and analysed are assumed to be a permanent structure in which wind load analysis on the building during various construction stages is not considered. Before wind loading analysis is carried out, wind behaviour and characteristics are also limited. The location of the building model is chosen at Peninsular Malaysia (Zone 1) as provided in MS1553 with a wind speed of 33.5 m/s.

1.5 Importance of the Study

The findings of this project would enhance the knowledges of structural engineers who are involved with wind load design on tall buildings based on various codes examined in this study. Besides, the conduct of the project would emphasise on the necessity of wind load being considered during the design of tall buildings. With wind load design taken into account for tall buildings, society is benefited when the safety of tall buildings has been secured under the critical condition due to wind-related disasters. Additionally, occupancy comfort can be ensured when vibration and deflection under serviceability limit state of tall buildings are mitigated or minimised.

With the comparative study of various codes for wind load design in this project, structural or wind engineers able to identify and choose the appropriate standard codes to be applied in building design. Furthermore, structural or wind engineers can understand the principles and theories for the differently used parameters and factors contributing to larger building responses for the various standard codes. Accordingly, proper applications of necessary and appropriate rules and regulations in wind load design codes can be accomplished during the design of high-rise buildings.

For the model analysis conducted in this study using software, structural or wind engineers would be acknowledged with the different building responses in the ultimate condition for member forces in the shear wall and support reaction of different building models. Well understanding of the building response of different building structures under wind effect would benefit them in studying wind behaviour on building structure.

1.6 Outline of the Report

This project report comprises of a total of five chapters. Chapters such as introduction, literature review, methodology, results, and discussion, and last but not least, conclusion and recommendations are encompassed.

In chapter 1, it depicts the general background of wind load and wind load parameters or factors, as well as the problems incurred due to wind-related disasters. Next, the aim and objectives of the project report have been identified. Besides, the scope and limitations of the study have also been clarified. The contribution of the study has also been determined. Ultimately, chapter 1 is ended with an organised outline of the report.

In chapter 2, the literature review on this project report is presented. Reviews of some studies by past researchers have been implemented. Topics such as disaster or damage caused by wind, wind characteristics, and behaviour and parameters affecting wind effects are discussed and revealed in detail.

In chapter 3, methodology, which describes the workflow of the study, is reviewed. Details or information about the methods, wind design codes, parameters of wind load, models development, and engineering software employed during the execution of this project report are clearly explained.

In chapter 4, results and discussion are described and interpreted. Results obtained through engineering software analysis are illustrated and presented. Discussion about the comparison of high-rise buildings response due to wind load based on different codes is written.

In chapter 5, conclusion and recommendations are conducted for this project report ultimately. Summary of the overall project and achievement of the aim and objectives of the report are determined. Also, suggestions or guidance are provided to aid in the implementation of subsequent related projects or researches.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter begins with the definition and explanation of wind load. Next, the identification of various wind-related disasters is reviewed. Besides, this chapter also includes the topic of along and across-wind loading. Next, the definition of high-rise buildings and parameters related to height in different codes are reviewed. Moreover, the topic of wind speed variation on high-rise buildings is discussed. Subsequently, wind drift and drift control are reviewed. Furthermore, wind pattern in Malaysia is assessed in this chapter. Additionally, several parameters of wind effects are evaluated. The parameters discussed are aerodynamic modifications of buildings, the shape of buildings, aspect ratio of buildings, interference effect, topographic effect, terrain condition, wind directionality, seasonal variation, and gust factor. Ultimately, a review of the comparison between codes is executed on the topics of basic wind speed, wind speed factors, external pressure, and internal pressure on buildings.

2.2 Wind Load

Wind load is a lateral load exerted on the surface of buildings in the way of the flow of wind. Far different from the static load and live load, wind load changes rapidly and vigorously, even much more significant effects are created when the same magnitude of wind load was applied gradually (Rajmani and Guha, 2015). The period for wind load to act on a building affects the categorisation of wind load into static or dynamic wind load. Wind load is considered as a static load when loading acts for an extended period while as a dynamic load for short term loading. However, the static load is considered instead of the dynamic load to represent the paramount value during the design stage. Dynamic response from wind load is usually contemplated during consideration of a relatively flexible structure (Hallebrand and Jakobsson, 2016). Dynamic load effect generated on buildings is amplified under the condition where wind fluctuation is mainly contributed by smaller eddies, which cause slender buildings to vibrate at or approximate to the natural frequency of the structure (Mendis, et al., 2007).

Vortex-shedding, which arises in the study of fluid dynamics, is a phenomenon applied to the study of wind load patterns and wind directions on building structure (bluff body) as illustrated in Figure 2.1. During the process of vortex shedding, vortices are formed on the crosswind side of the building, which results in crosswind excitation. Thus, magnification of amplitude of displacement tends to occur when the natural frequency of building approaching or corresponds to shedding frequency (Mendis, et al., 2007).

With turbulent wind flow, vortices (spiral flow) are developed. Significantly, vortices are considered during the study and analysis of buildings' vibration. As mentioned by Roy, et al. (2017), the symmetrical distribution of vortices occurs under the condition of relatively low wind speed (Figure 2.2a). The force equilibrium is achieved when vortices exist on both transverse sides of a structure acting in opposing direction. Hence, no vibration of the building is incurred across wind direction in which the crosswind effect can be neglected when only low wind speed is considered.

On the contrary, under the condition with high wind speed, uneven distribution of vortices is incurred due to alternative generation of low-pressure zones on building's crosswind sides, as shown in Figure 2.2b (Roy, et al., 2017). Across-wind vortices applied mainly on one side of the structure body has a significant effect on a tall building, especially with higher slenderness ratio. Tall or slender building tends to shudder when crosswind impulse becomes paramount as compared to alongwind impulse with more significant across-wind acceleration than along-wind acceleration (Fu, 2018).

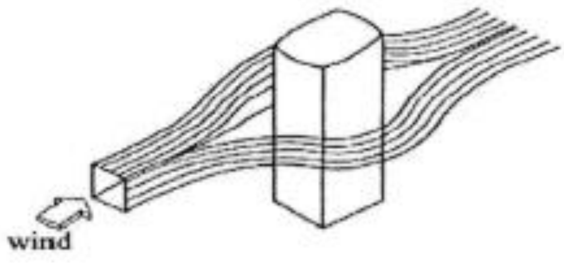

Figure 2.1: Simplified Wind Flow (Roy, et al., 2017).

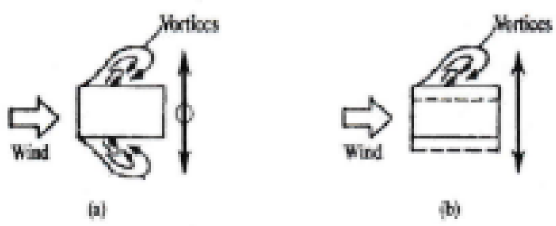

Figure 2.2: Vortices under Distinct Magnitude of Wind Velocity (a) Vortices during Gradual Wind Speed (b) Vortices during Rapid Wind Speed (Roy, et al., 2017).

2.3 Wind-Related Disaster

The design of the building's structure also needs to withstand the external loads caused by nature despite sustaining its self-weight, live load, and dead load. Natural wind-related disasters such as windstorm, hurricanes, and tornadoes are occurrences which contribute significant wind loading on building structure. Consideration of wind load in buildings design is a must to minimise the catastrophic destruction to an acceptable limit in which happenings of buildings failure and loss of life could be prevented (Dagnew, et al., 2009). The tremendous amount of damages and failures of building structures have happened in Malaysia historically, which are mainly triggered due to high wind speed. Therefore, the analysis of wind loads and wind effects are significantly required on building structure, especially for high-rise buildings.

As reported by Tan (2020), the incident of sandstorm due to strong wind has occurred at Gurney, Penang, as shown in Figure 2.3. This happening had caused toppling over of trees and tearing off of zinc roofs of houses nearby. Besides, during the monsoon season from 24th September to the beginning of November, Peninsular Malaysia's west coast and northern Borneo states of Sabah and Sarawak were struck by a torrential downpour. The incident had triggered severe damage to infrastructure and flooding (Xinhua, 2019). Besides, waterspout (tornadoes over water) was reported in Penang Island, as illustrated in Figure 2.4. Wind speed of approximately 80 mph, which tossed scattered debris a few hundreds of meters into the air, had caused damage to approximately 50 buildings nearby (Cappucci, 2019). According to Hamizah (2010), windstorm, as the top 100 natural disasters in Malaysia, had troubled 40000 people in east Peninsular Malaysia on 6th November 2004 in which a lot of casualties and damages have been incurred. At Bukit Mertajam in Seberang Perai on 16th August 2004, the issue of roofs falling off from an apartment that caused catastrophic damages to 20 vehicles had been recorded (Hamizah, 2010), as shown in Figure 2.5.

Figure 2.3: Sandstorm Incident in Penang (Tan, 2020).

Figure 2.4: Massive Waterspout Happened in Malaysia (Cappucci, 2019).

Figure 2.5: Roofs Collapsing on Vehicles (Hamizah, 2010).

From January 2009 till June 2012, numerous damage cases that are related to wind disasters in Malaysia for each state as the concern of wind effect towards buildings are trivial previously. Figure 2.7 shows the statistical value of disaster cases due to the windstorm in Peninsular Malaysia. Strikes of gusty wind in Seremban in 2016 had triggered the collapse of ceiling parts of D'S2 Mall PKNS Complex building (Hamzah, et al., 2018). Besides, Macalister Road Tragedy, as illustrated in Figure 2.6, happened in Penang on 3rd June 2013 in which din-shaped wall attached to 21-storey UMNO Tower with a height of approximately 100 m had collapsed during a thunderstorm. The tragedy that occurred had led to two casualties and five injuries. Wind speed of 17.5 m/s is indicated during the happening of the tragedy in Penang. The wind speed was considerably lower as compared to basic wind speed suggested in MS 1553: 2002 (Ramli, et al., 2015).

Figure 2.6: Macalister Road Tragedy (Chu, 2019).

Another incident due to the natural disaster of strong wind had been recorded in Kota Kinabalu. The strong wind had struck the housing area near villages and island, which caused severe damage to the roofs, even some roofs were torn off (TheStar, 2019). As mentioned by Ramli, et al. (2015), the occurrence of windstorm has engendered 80% of the cases related to damaged roofing systems. Scattered debris from the damaged roofing system of a relevant building causes impacts on surrounding buildings, which has resulted in the evaluation of the impact loads due to wind disaster during design.

According to Ramli, et al. (2014), material and connection of roof sheeting are two significant factors contributing to the failure of the roof system. These structural elements must be designed suitably to have sufficient capacity to withstand wind speed as far as 32.5 m/s following MS 1553: 2002.

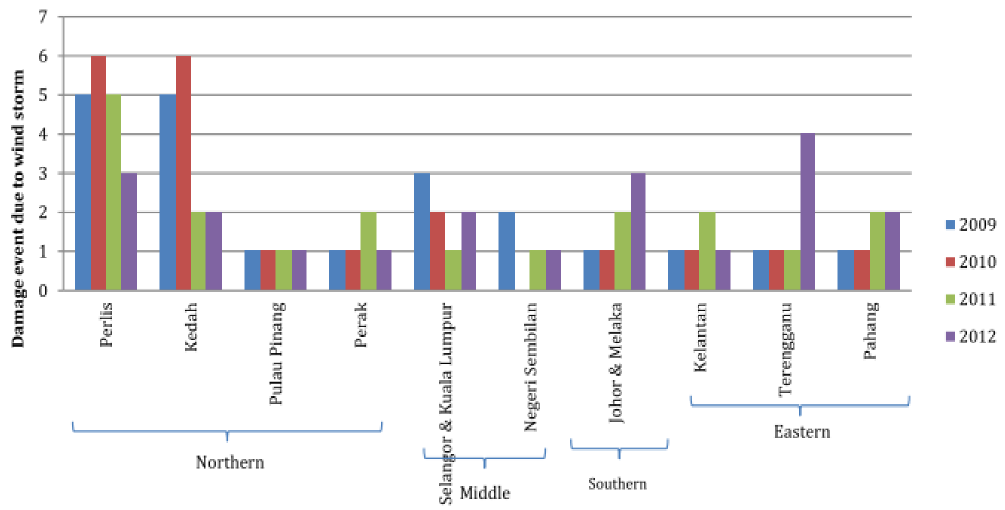

Figure 2.7: Statistics of Destruction Caused by Windstorm in Peninsular Malaysia on Jan 2009-June 2012 (Hamzah, et al., 2018).

2.4 Along and Across-Wind Loading

When wind flows in a particular direction to or near a building, wind flow is separated, and vortices are generated in the wake region during turbulent flow. Buildings structure, especially of high-rise buildings, has distorted wind flow, which complicates the study and analysis of wind flow patterns. Hence, inconsistent wind force magnitude is determined to act on buildings structure (Hallebrand and Jakobsson, 2016). Additionally, vigorous wind pressure fluctuations instead of steady flow occurred on the building's surface have caused aerodynamics loads to act on building structure (Mendis, et al., 2007).

Wind-induced fluctuations are generally divided into three different modes of action (two lateral modes & one rotational mode): along-wind, across-wind, and torsional modes, as shown in Figure 2.8 (Mendis, et al., 2007). Typically, across-wind and torsional motion are determined to be more critical compared to along-wind motion during the design of a tall building.

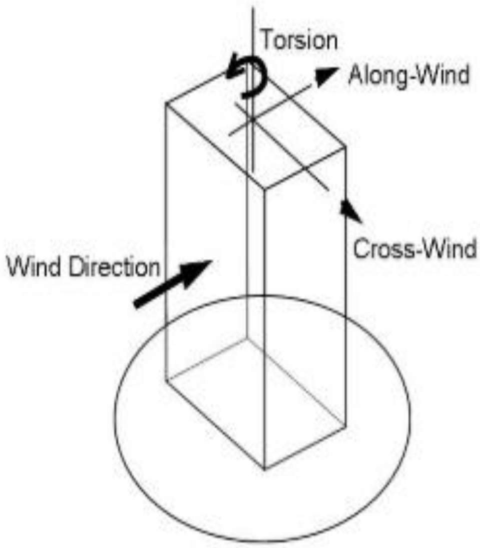

Figure 2.8: Orientation of Three Distinct Wind Modes (Mendis, et al., 2007).

Along-wind force is also termed as drag force with an along-wind response interpreted as the building's response due to wind-buffeting (Gordan, et al., 2014). The along-wind load consists of an intermediate component, which is contributed by moderate wind velocity action and a random component on account of wind velocity deviation from the average value (Mendis, et al., 2007). Primarily, along-wind motion is induced by pressure variations on the windward and leeward direction of buildings, which results in swaying motion of buildings parallel to wind flow direction (Ilgin and Gunel, 2007). Along the direction of along-wind motion, buildings tend to deflect more and experience significant horizontal wind force (Ilgin and Gunel, 2007). In the along-wind direction, dynamic response of buildings can be estimated by the gust factor approach under the condition where the effects of surrounding skyscrapers and terrain (Mendis, et al., 2007).

Across-wind load is termed as a force acting perpendicularly to the direction of the wind (Mendis, et al., 2007). Across-wind flow is resulted from the splitting of wind flow due to the blockage of air movement by buildings. Swaying motion of buildings, which is orthogonal to the wind flow direction, is determined for across-wind motion. Along wind force has its significant effect on structure with height up to 150 m and long-orientated body while across wind load is governed for all buildings with a short-orientated body (Aiswaria and Jisha, 2015). According to Gu and Quan (2004), a wind tunnel test conducted on the Jin Mao Building has determined the factor of 1.2 times of maximum acceleration of across-wind direction to along-wind acceleration. Thus, the significance of across wind response of buildings for high-rise buildings is affirmed.

Despite two lateral wind modes, a torsional motion that is seldomly evaluated does exist. The occurrence of torsional motion is mainly due to the discrepancy of pressure dissipation on each surface of buildings. The torsional moment is induced when significant variation exists between the elastic centre and aerodynamic centre of the building structure (Amin and Ahuja, 2010). Torsional responses become momentous for building acquired with the characteristic of shape irregularity in which asymmetrical flow is generated (Günel and Ilgin, 2014). Various factors such as angled wind direction, uneven approaching flow, interference effect by surrounding buildings, and irregular shape of buildings itself with an inconsistent centre of rigidity are evaluated to produce uneven dispersal of wind pressure on buildings (Dagnew, et al., 2009).

2.5 Definition of High-Rise Buildings

An accurate description of tall buildings is not recognised, and even standard features of buildings such as the number of floors or height of structure are challenging to be termed in considering a building structure as high-rise building (Roy, et al., 2017). The categorisation of the low, medium, and high-rise buildings is not specifically ranged into a certain height (m) and the number of floors in standard codes for wind load design. Besides, high-rise buildings are simply defined as buildings which have a remarkably taller than typical buildings or have higher slenderness to appear as tall building (Hallebrand and

Jakobsson, 2016). Moreover, through structural analysis, a high-rise building is generally defined when a significant lateral force is majorly considered into design. Safety design of a tall building is achieved through the application of lateral stability systems such as the shear wall system and bracing system in resisting horizontal forces caused by wind actions (Fu, 2018). According to Craighead (2009), a tall building is defined to have a height above 22.5 m, as stated in Fire Safety Code in the United States. Height of building is specified as the accessibility of buildings for certain height is considered in fire prevention measures of fire safety authority.

2.6 Parameters Related to Buildings Height

As stated by(CEN, 2004), in European Standard (EN 1991-1-4:2004), buildings are parted according to the 7, h (building's height)/ b (crosswind width of structure) of buildings into three categories which are h is smaller than b, h is within b and 2b, and h is larger than 2b. For each part distributed, each different wind pressure profile is referred to obtain wind pressure on along buildings surface from ground. However, the maximum building height of 200 m is stated for the application of EN 1991-1-4:2004 in wind load design.

British Standard (BS 6399-2:1997) also divides buildings into parts according to aspect ratio as implemented in EN 1991-1-4:2004 for the calculation of wind load (British Standard Institution (BSI), 2002). Method of division by parts applied results in higher base shear and moment produced by EN 1991-1-4:2004 and BS 6399-2:1997 compared to AS/NZS 1170.2:2011 (Weerasuriya and Jayasinghe, 2014). BS 6399-2:1997 also has the same limited height of buildings up to 200 m as EN 1991-1-4:2004 (Weerasuriya and Jayasinghe, 2010). However, some parameters or factors obtained for gust peak factor and dynamic augmentation factor are limited to buildings height of 300 m in BS 6399-2:1997.

In AS/NZS 1170.2:2011, a torsional response is considered for rectangular buildings with height more than 70 m in the computation of wind action (Joint Standards Australia, 2011). Furthermore, buildings with natural frequency smaller than 1 Hz are subjected to dynamic response (Holmes, 2001). The height of buildings is also limited to 200 m as specified in AS/NZS 1170.2:2011, clause 1.1. As clarified by Joint Standards Australia (2011), three wind responses are anticipated, and wind tunnel test is carried out for the building with a height higher than 200 m under the condition with a natural frequency.

In EN 1991-1-4:2004, span ratio, d (windward depth of structure)/ h (height of the wall) is applied to categorise external pressure coefficient, C_{pe}. According to CEN (2004), C_{pe} values are tabulated according to h/d ratio and each zones (A, B, C, D & E) of vertical walls of building as shown in Table 2.1. BS 6399-2:1997 groups C_{pe} values for vertical walls with span ratio of less or equal to 1 and more or equal to 4 as shown in Table 2.2 (British Standard Institution (BSI), 2002). In AS/NZS 1170.2:2011, C_{pe} values are grouped differently for windward and leeward direction into tables for walls and different type of roofs. Buildings height of 25 m is used to differentiate each C_{pe} values for windward wall whereas degree, roof shape and d/b ratio are used for leeward wall (Joint Standards Australia, 2011).

Table 2.1: Suggested External Pressure Coefficients for Walls (CEN, 2004).

Zone	A		B		D		D		E	
h/d	$c_{pe},10$	$c_{pe},1$	$c_{pe},10$	$c_{pe,1}$	$c_{pe},10$	$c_{pe},1$	$c_{pe},10$	$c_{pe,1}$	$c_{pe},10$	$c_{pe},1$
5	-1.2	-1.4	-0.8	-1.1	-0.5		+0.8	+1.0	-0.7	
1	-1.2	-1.4	-0.8	-1.1	-0.5		+0.8	+1.0	-0.5	
≤ 0.25	-1.2	-1.4	-0.8	-1.1	-0.5		+0.8	+1.0	-0.3	

Table 2.2: External Pressure Coefficient for Walls (British Standard Institution (BSI), 2002).

Vertical wall force	Span ratio of building		Vertical wall force		Exposure case	
	D/H ≤ 1	D/H ≥ 4			Isolated	Funnelling
Windward (front)	+0.85	+0.6	Side	Zone A	-1.3	-1.6
Leeward (rear)	-0.5	-0.5		Zone B	-0.8	-0.9
				Zone C	-0.5	-0.9
NOTE: Interpolation may be used in the range 1 < D/H < 4. See 2.4.1.4 for interpolation between isolated and funnelling						

Buildings with a ratio of h /d higher than 4 are clarified to be slender in EN 1991-1-4:2004 (CEN, 2004). Moreover, structural factor, $c_s c_d$ is considered in EN 1991-1-4:2004 for the computation of dynamic response. EN 1991-1-4:2004 groups $c_s c_d$ values according to the specific height of buildings and c_d (dynamic coefficient) is also computed with equations under several different conditions. For instance, $c_s c_d$ is assumed as 1 for buildings with elevation level lower than 15 m and structurally framed buildings with height less than 100 m and h < 4d (CEN, 2004). As mentioned by British Standard Institution (BSI) (2002), dynamic augmentation factor, C_r is applied to compute overall load for buildings subjected to mildly dynamic response. C_r value is obtained through graph as shown in Figure 2.9 which is limited to C_r value of 0.25 and buildings height of 300 m. According to Holmes (2001), C_r value is attained through other standards or standardised references for buildings with height exceeding 300 m. In AS/NZS 1170.2:2011, dynamic response factor, C_{dyn} is obtained based on natural frequency in which C_{dyn} is assumed as 1 for natural frequency equal to or more than 1 Hz.

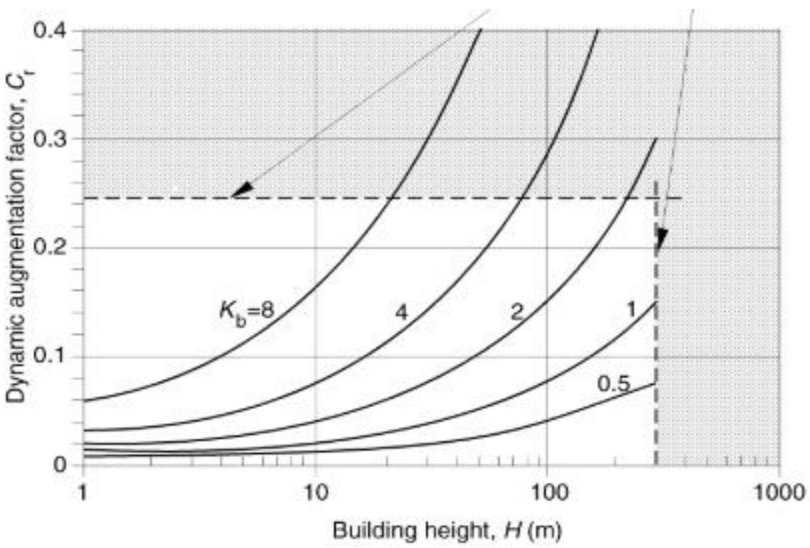

Figure 2.9: Dynamic Augmentation Factor (British Standard Institution (BSI), 2002).

2.7 Wind Response Variation on High-Rise Buildings

Wind load effect is significantly vital for high-rise buildings as wind loads acting on buildings rise substantially with an increment of the height of buildings, as illustrated in Figure 2.10. Most massive moments originated at the base of a building are necessary to be considered into the design and stiffness design of a building is minimised following increment of a building's height (Hallebrand and Jakobsson, 2016). Meanwhile, the wind speed also increases with the increment of wind action as the square of the wind speed, which is only applicable for low and medium-rise buildings. On the other hand, by considering the resonant building response due to dynamic load, wind speed is varied to power larger than two following buildings height (Holmes, 2001). The relationship of wind pressure and the square of wind speed corresponding to elevation presented in Figure 2.11 has indicated a proportional increment (Alaghmandan and Elnimeiri, 2013). As mentioned by Rajmani and Guha (2015), the trend of increasing wind speed is indicated in a curved line varying from zero at the ground surface to an extreme at an elevation above the ground.

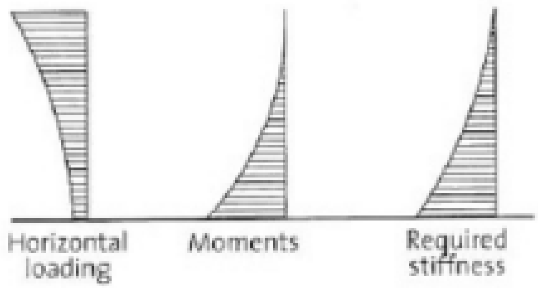

Figure 2.10: Wind-load, Moment and Stiffness Illustration for a Tall Building (Hallebrand and Jakobsson, 2016).

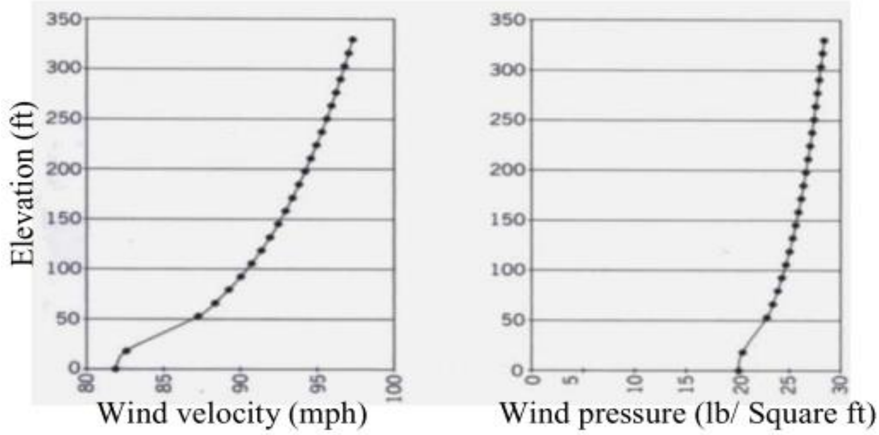

Figure 2.11: The Relationship between Height, Velocity and Pressure (Alaghmandan and Elnimeiri, 2013).

2.8 Wind Drift & Drift Control

Wind drift has the definition of lateral displacement of one level of multi-storey buildings relative to above or below level caused by horizontal wind load with magnitude increased following the building's height (Rahman, et al., 2014). Additionally, inter-storey drift, which describes the lateral displacement between floors, is another term usually interpreted for buildings response (Günel and Ilgin, 2014). According to Hadihosseini, Hosseini and Hosseini (2014), a significant effect of wind drift is evaluated on either structural elements or non-structural elements and adjacent structures.

Drift index is the value computed, which has the definition as the ratio of lateral displacement between floors to adjacent floor height (Δ/h). Peak inter-storey drift index and wind drift index are important parameters considered for the evaluation of lateral stability and stiffness of the structural system of buildings (Arum and Akinkunmi, 2011). For wind load design of high-rise buildings, the index value of 1/400 to 1/500 is generally applied for both wind drift and inter-storey drift index (Günel and Ilgin, 2014). Wind drift deflection limits are adopted to minimise the destruction of façade cladding, partitions, and interior finishes of buildings. Besides, different wind drift index is resolved for various design codes to reduce perceptible movement of buildings and to restrict lateral displacement of a building which is also termed as P-Delta effects (Weerasuriya, et al., 2010). Inter-storey index is contributed by two components, which are shear and displacement between storeys. A shear that is labelled as "racking drift" represents the component of relative motion of adjoining floors measured parallelly, whereas displacement or "chord" drift involves the determination of rotation angle from chord rotation between adjacent floors (Mendis, et al., 2007).

According to Awida (2010), high-rise buildings which have higher slenderness ratio tend to have more extensive wind drift index compared to low and medium-rise buildings as illustrated in Figure 2.12. In the analysis conducted by Awida (2010), tall buildings are defined as buildings with a slenderness ratio of greater than 6.0 while the medium building has a slenderness ratio within 3.0 to 6.0 and low-rise building has a slenderness ratio of less than 3.0.

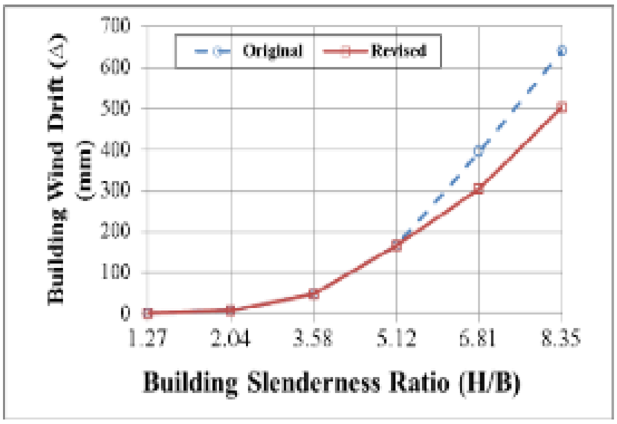

Figure 2.12: Effect of Slenderness Ratio on the Wind Drift (Awida, 2010).

Drift or deflection caused by lateral wind load on tall buildings has led to the study of various structural approaches by researchers to maintain stiffness and strength of building structures. Mitigation of drift control is introduced to sustain or minimise lateral deflection of buildings to ensure the functionality of non-structural components in building structures, to avoid excessive cracking of concrete and to prevent the reduction of the stiffness of building (Islam, et al., 2012).

To reduce the drift effect, structural components such as frame action, shear wall, and dual-system are commonly used to diminish lateral deflection (Arum and Akinkunmi, 2011). It is mentioned that the shear wall is applied to optimise the stiffness of the building's structure, and the determination of the location of the shear wall has a stimulating effect influencing axial load in columns, bending moment, and shear force in beams (Rajoriya and Uttam, 2016). The utilisation of shear walls is also proven by Chittiprolu and Kumar (2014), with the result indicating that story drift is reduced when the shear wall system is applied to building structures. Besides, structural components of beams and columns also contribute to drift control. The increase of moment of inertia (sizes) of beams and columns benefits the reduction of drift effect (Islam, Siddique and Murshed, 2012). The research conducted by Rahman, Fancy and Bobby (2015) also mentioned that the lower drift index is obtained for beams and columns with a larger dimension, as shown in Figure 2.13. According to the findings by Kevadkar and Kodag (2013), the application of steel bracings as structural components of the building also assists in resisting lateral wind action.

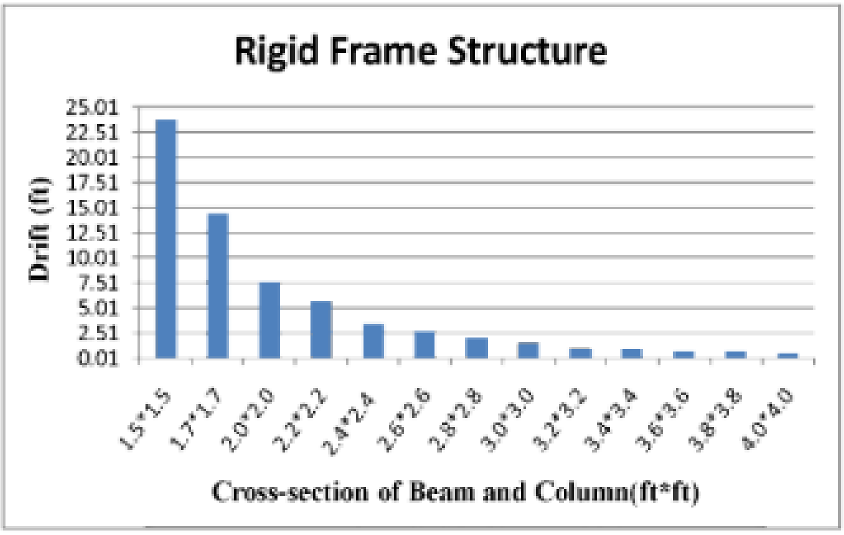

Figure 2.13: Variation of Drift with Dimension of Beam and Column (Abdur et al., 2012).

2.9 Wind Pattern in Malaysia

Due to the location of Malaysia closer to the equator, wind flow pattern across the nation is mainly governed by two significant monsoons which are the flow of Southeast and Northwest monsoon with two shorter period of inter monsoon seasons as their transition. The frequent occurrence of a thunderstorm is usually reported during inter monsoon seasons (Ramli, et al., 2014). During the southwest monsoon, wind velocity determined is below 7.7 m/s, which is equivalently lower compared to northeast monsoon with steady wind speed within the range of 5.2 to 10.3 m/s. Moreover, wind velocity of 15.4 m/s is reached for wind flow across the east coast states of Peninsular Malaysia (Nizamani, et al., 2018). According to Nizamani, et al. (2018), the highest average daily wind speed of 3.8 m/s is reported at Mersing in Johor state while 41.7 m/s is the most considerable extreme wind velocity recorded at Kuching, Sarawak in the year of 1992 on 15th September. According to the study of wind speed attained in Peninsular Malaysia during the monsoons period from 1999 to 2008 by Satari, et al. (2015), a smaller difference in wind speed is recorded on the east side of Peninsular Malaysia compared to the west side. In east Peninsular Malaysia, the average wind speed with the range of 7.53 to 9.49 m/s is obtained, whereas values in the range of 7.02 to 9.54 m/s are recorded in West Peninsular Malaysia. Additionally, coastal areas in Peninsular Malaysia are determined to provide a wind speed of less than 8.5 m/s from the years 1999 to 2008 (Satari, et al., 2015).

The determination of basic wind speed, which is considered in wind load design, is decided through Gringorten Method Analysis for a recurrence interval of 50 years, according to MS 1553:2002 (Nizamani, et al., 2018). As shown in Figure 2.14 below, basic wind speed is distinguished for Zone I, which represents the inland region, and Zone II, which represents the shoreline region (Hamzah, et al., 2018).

Figure 2.14: Basic Wind Speed throughout Peninsular Malaysia based on Zones (Department of Standards Malaysia, 2007).

2.10 Parameters affecting wind effects

Despite the satisfaction of the ultimate limit state for buildings to sustain wind load imposed, it is necessary to consider serviceability issues induced by wind actions. The execution of building envelope during wind motion and comfortability of buildings occupants are two main crucial parameters needed to be evaluated (Amin and Ahuja, 2010). Through experimental study and analysis implemented, factors of wind effects on the tall building have a significant influence on building response. Researchers have investigated and examined various factors affecting the building's response due to wind actions such as aerodynamic modifications of buildings, the shape of buildings, aspect ratio of buildings, interference effect, topographic effect, terrain condition, wind directionality, seasonal variation, and gust factor.

2.10.1 Aerodynamic Modifications of Buildings

Wind engineers often consider the architectural and structural systems in mitigating and minimizing the wind effects on tall buildings (Alaghmandan and Elnimeiri, 2013). Under architectural mitigations, modifications on the aerodynamics behaviour of buildings are mainly evaluated. Aerodynamic modifications are implemented on tall buildings to moderate wind motion and alter wind flow. Example of modifications applied includes remodelling of cross-sectional shape or corner geometry of buildings and applying different orientation of openings or sculptured tops of high-rise buildings (Neethi and Joby, 2018). Generally, two groups of aerodynamic modifications are categorised, which are minor modifications and significant modifications. For minor modifications, moderation of buildings corner is predominantly involved. On the other hand, significant modifications focus on improvement for the architectural concept of buildings, which comprises of higher complexities (Amin and Ahuja, 2010).

Based on findings by Alaghmandan and Elnimeiri (2013), various aerodynamic modifications are applied on 73 high-rise buildings constructed by 2012 with an elevation of more than 300 m, as shown in Figure 2.15. As reported by Ilgin and Gunel (2007), a 25% depletion of the base moment for Taipei 101 is achieved through the application of corner modifications. Besides, with 10% building width chamfered for Taipei 101, 40% and 30% reduction in along-wind and across-wind response respectively are accomplished. Modelling on 150 m tall buildings performed by Neethi and Joby (2018) through ANSYS software has revealed the reduction effect of aerodynamic modifications on buildings to drag coefficient, drag force, and moment about the foundation of buildings. It is concluded that setback, tapering, and sculptured tops are intelligent design for high-rise buildings in reducing wind excitation (Amin and Ahuja, 2010).

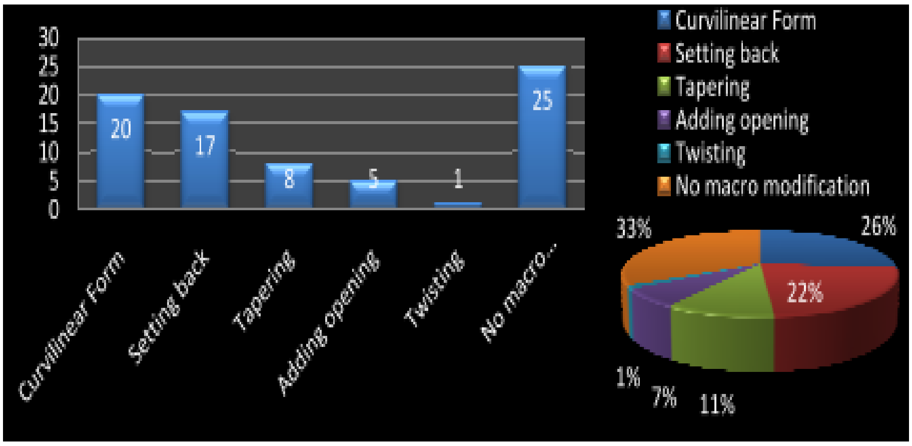

Figure 2.15: Amount and Rate of Tall Buildings Applying Geometry & Form (Alaghmandan and Elnimeiri, 2013).

2.10.2 Shape of Buildings

Wind load effect on flexible high-rise structure is controllable with the practical shape of buildings selected during design. According to Kiran Kumar and Dhiyaanesh (2018), it is determined that wind response is mitigated in higher efficiency for the rounded shape of buildings than the angular shape. The condition can be explained as wind is redirected to a smaller angle due to the aerodynamic shape of buildings, which allows smooth wind flow. Significance of the shape of buildings is affirmed by Mashalkar, Patil and Jadhav (2015) during their research done on the examination of wind effect by the various shape of buildings (I, T, L, and C shaped). As a result, the I-shaped building provides less storey drift and lateral deflection compared to other shapes, and the symmetrical characteristic of the shape is considered as the factor contributed to the reduction of wind response.

In the study of building configurations as a significant architectural modification on wind load, Roy, et al. (2017) mentioned that the wind pressure coefficient is maximum for cubic-shaped high-rise buildings while minimum value is obtained for circular shape and swastika shape. A square plan shape generates less drag force compared to other shapes. Figure 2.16 has presented the distinct cross-sectional shapes of tall buildings with the same area of the plan are taken into consideration during the analysis carried out by Roy, et al. (2017). According to Kulkarni and Muthumani (2016), the application of circular and elliptical shapes in designing buildings benefits the reduction of wind pressure and drag force. Figure 2.17 has illustrated the various magnitude of drag force generated for each different geometry of buildings.

As stated by Sazzad and Azad (2015), the area of exposure is considered as a parameter in affecting wind response on various shapes of buildings. Additionally, the factor of slenderness ratio is also reviewed in the selection of the shape of buildings as proven by the research carried out by Hemanthkumar and Kiran (2017) on comparing models with distinct shape and lateral length ratio.

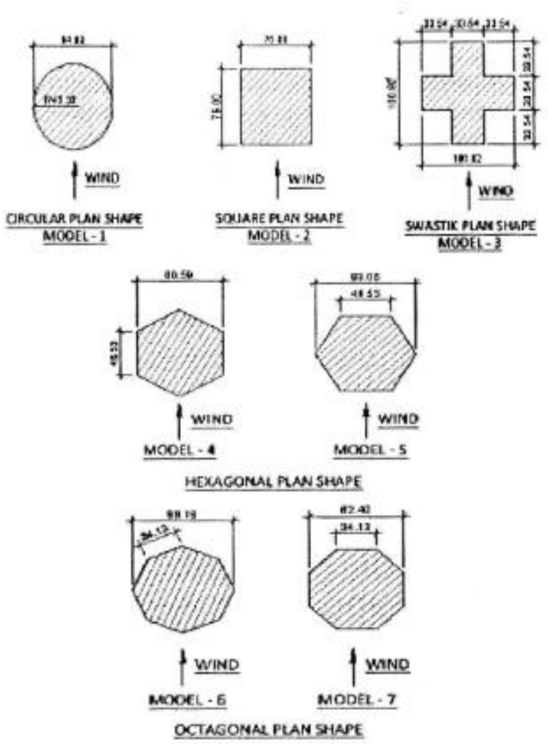

Figure 2.16: High-rise Building Geometry with Distinct Cross-sectional Shapes with the Identical Plan Area (Roy, et al., 2017).

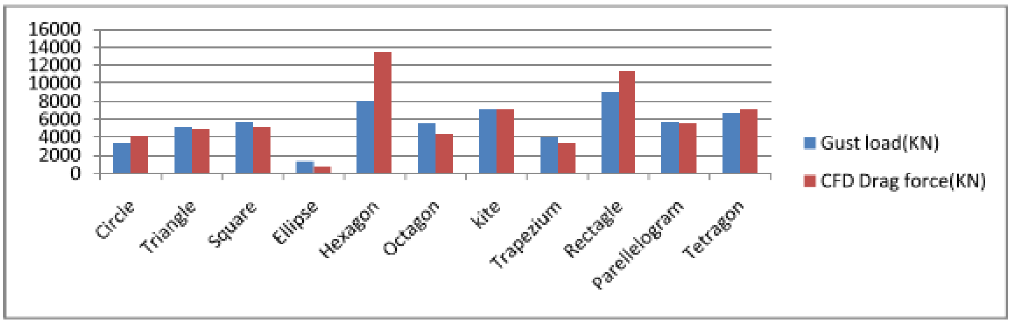

Figure 2.17: Variation in Wind Forces for All Shapes (Kulkarni and Muthumani, 2016).

2.10.3 Aspect ratio of Buildings

The slenderness ratio (h/b) of a building itself is an important parameter taken into consideration in mitigating wind response on tall buildings. In the case study of Awida (2011), towers with a distinct slenderness ratio of 4.77 (Tower B) and 8.60 (Tower A) are examined, and the buildings incur results of higher wind drift, higher wind acceleration, a higher torsional base moment of buildings as shown in Table 2.3 with larger slenderness ratio. Despite slenderness ratio (vertical aspect ratio), the plan aspect ratio of tall buildings obtained by dividing the length of buildings with base width is also analysed to understand the behaviour of building due to wind motion (Shelke and Kuwar, 2018). Under wind load, storey displacement of buildings increases with the increment of aspect ratio. Besides, axial forces acting on columns are determined to increase following the rises of aspect ratio (Shelke and Joshi, 2019). According to Avini, Kumar and Hughes (2018), a decrease of mean drag has resulted from the minimization of b/d or 2h/b where b and d are defined as plan dimensions arranged across-wind and along-wind direction.

Table 2.3: Outcomes of Wind Tunnel Test (Awida, 2011).

Tower	Value Input				Outcomes				
	Fundamental Period (Sec)				Extreme Base loads (MN, m)				Acceleration (milli-g)
	T1	T2	T3	FX	FY	MX	MY	MZ	10 years
A	4.0	3.0	2.8	6.1	1.9	125	392	22.3	18.5
B	2.8	2.5	1.8	2.0	2.6	174	126	7.40	6.8

2.10.4 Interference effect

The interference effect is the phenomenon that occurs when the magnitude of wind forces and pressures acting on principal building manifested due to the existence of adjacent structures (Gajjar, Jhumarwala and Umravia, 2018). As stated by Lam and Zhao (2017), the ratio of the typical value of wind effect under the interference effect to the corresponding value for the isolated condition is known as an interference factor (IF). The interference effect can either reduce or increase wind loads and pressures on tall buildings (Gu and Xie, 2011). When IF is greater than 1, wind response on the principal building is increased and vice versa. Besides, it is mentioned that the shielding effect is related to the IF value. With smaller IF values than 1 in which wind load is reduced, the phenomenon of shielding effect is verified (Cho, et al., 2004). Shielding effect is another term introduced when wind load is depleted under the condition where the interference effect is considered (Avini, et al., 2018). Through pressure measurement experiments conducted by Hui, et al. (2013), it is determined that negative pressure determined for buildings with interference effect surges 50% higher than the buildings under an isolated condition.

According to Kheyari and Dalui (2015), several factors such as various shapes and sizes of buildings or surrounding buildings, orientations of buildings, terrain conditions, and wind direction have a high impact on IF value. The interference distance between interfering buildings and principal buildings, as shown in Figure 2.18, is also the factor affecting the interference effect according to You, Kim and You (2014). Wind angle of 60° and 90° give higher interference factors on interfering buildings due to the generation of more vortices and separation of wind flow (Kheyari and Dalui, 2015). Through experimental analysis, interfering buildings with half-height have little effect on the mean wind pressure or load compared to interfering buildings with full-height (Dagnew, et al., 2009). Through the study on the interference effect of 2 or 3 buildings by Gu and Xie (2011) as illustrated in Figure 2.19, it is discovered that two interfering buildings incur a more substantial shielding effect compared to a single interfering building. Besides, either higher or wider interfering buildings also benefit the shielding effect on the principal building. In term of the arrangement of buildings in a row, the diamond pattern chosen as shown in

Figure 2.20 in arranging buildings in a row has a magnifying effect on mean wind response on buildings at the majority of wind angles compared to the parallel pattern of arrangement (Lam and Zhao, 2006).

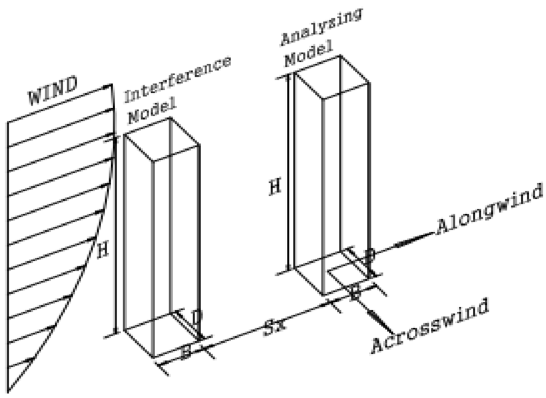

Figure 2.18: Relationship between Test Model and Interference Distance (You, Kim and You , 2014).

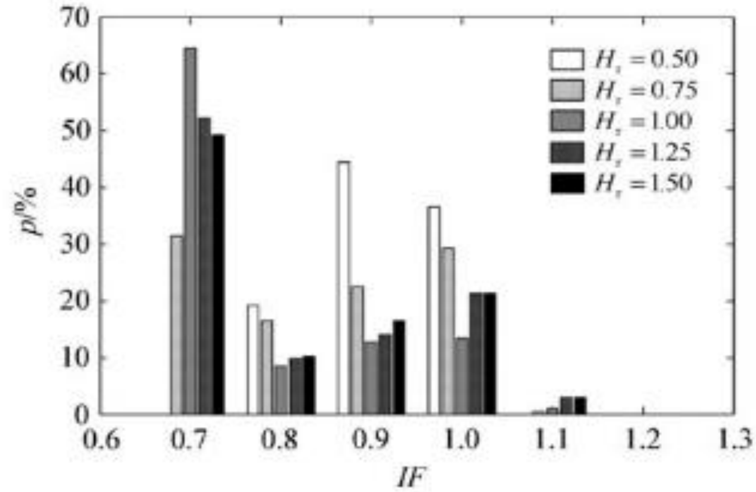

Figure 2.19: Statistical Results of Mean IFs for Various $H_r s$ (Gu and Xie, 2011).

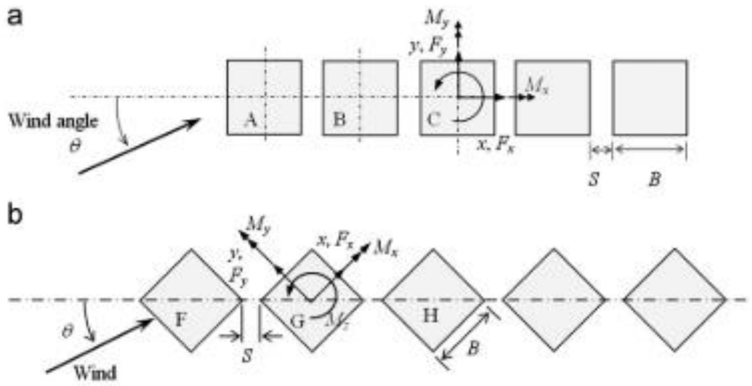

Figure 2.20: Wind Loads and Wind Orientation for a Row of High-rise Buildings: (a) Parallel Pattern: and (b) Diamond Pattern (Lam and Zhao, 2006).

2.10.5 Topographic effect

Topographic effect acts as an essential parameter in affecting wind response on tall buildings. According to Maharani, Lee and Lee (2009), topographic conditions such as crests of hills, ridges, and escarpments accelerate wind speed and alters typical wind speed profile. Topographic features listed obstructing and accelerating the wind flow, which leads to the consequence of magnified wind pressure acting on buildings surrounding the regions (Ngo and Letchford, 2008). As stated by Holmes (2001), different alterations of wind flow patterns following various topographic features, as illustrated in Figure 2.21, have been interpreted and related to the speed-up effect on wind flow. Several factors, such as hill shape factor, the distance of structure factor, and height of building factor, are parameters discovered to influence the topographic factor (Maharani, et al., 2009). As mentioned by Hamzah, Usman and Omar (2018), wind speed is increased when wind flows through mountain ridges. Besides, the occurrence of the venturi effect, which leads to an increase of wind velocity, is discovered when wind flow is channelled into canyons.

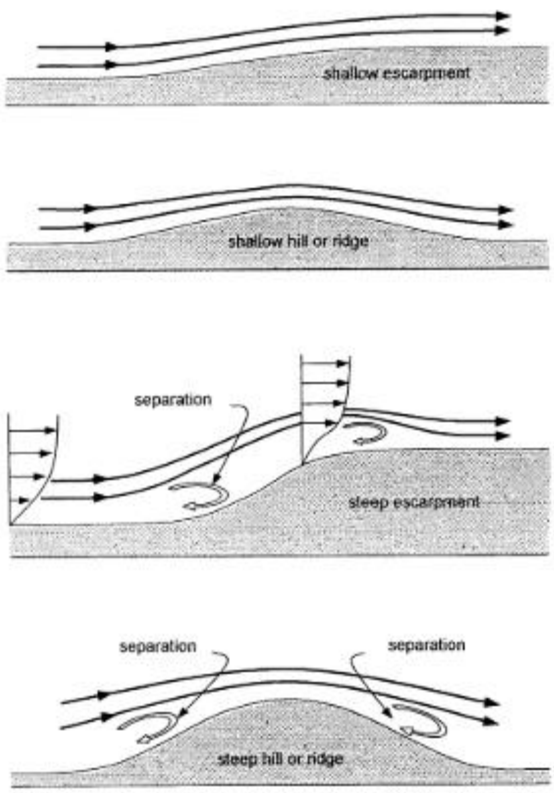

Figure 2.21: Flow Over Shallow and Steep Topography (Holmes, 2001).

2.10.6 Terrain condition

Various terrain categories are investigated in determining the wind response on tall buildings. Interaction of wind flow with rough terrain generates lower wind speed while higher wind speed is incurred due to smooth terrain. As stated by Ellison and Rutz (2015), the roughness of terrain, landscaping, or developed environment have significant impacts on wind response. Additionally, wind condition is studied through the term of roughness length, which is clarified as the height above ground level, where zero wind speed is obtained conceptually (Okafor, et al., 2017).

Lower variation of mean pressure and force coefficient on suburban terrain than open terrain has been discovered by Chitra, Harikrishna and Selvi (2017). Rough terrain with mountains and hills or built-up terrain around the principal buildings is verified to cause a reduction in wind strength and distort the wind direction (Li, et al., 2017). Experimental analysis carried out by Ahmed, et al. (2015) has shown the result of higher deflection obtained for over 30 stories of the high-rise buildings on exposed open terrain compared to other terrains with closely spaced or high-closely spaced obstructions. Moreover, Okafor, et al. (2017) mentioned that wind shear is governed by roughness drag through the irregular surface of the terrain. On the contrary, governance of wind shear by viscosity is determined on a smooth or flat surface. The inversely proportional relationship of wind speed and roughness coefficient of the surface, as shown in Figure 2.22 has been obtained through correlational analysis (Laban, et al., 2019). In order to interpret the roughness coefficient for particular terrain accurately, the application of geographic information system (GIS) is recommended due to its capability of eliminating the uncertainty and ambiguity accounted (Ellison and Rutz, 2006).

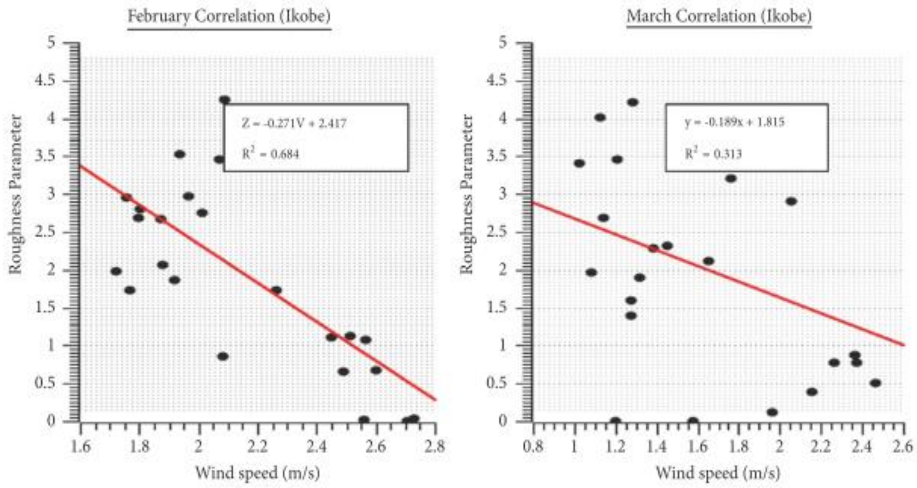

Figure 2.22: Correlation of Wind Speeds and Roughness Coefficient (Laban, et al., 2019).

2.10.7 Wind Directionality

Wind directionality factor, K_d is defined as a form of reduction factor for wind load with probability less than 100 per cent for the coincidence of worst aerodynamic design of buildings with critical wind flow direction (Laboy, et al., 2013). According to Habte, et al. (2013), K_d is a function comprised of the type of wind storm, specific wind climate in a geographical location, sort of wind effect and wind position pattern. K_d is determined to be larger in regions which are vulnerable to hurricane compared to non-hurricane areas (Habte, et al., 2015). Isyumov, Ho and Case (2014) also make affirmation on the higher value of K_d ($K_d = 0.9$) interpreted for the area which is prone to the occurrence of a hurricane. Likewise, definition of K_d as the proportion of the N year MRI (mean incurrence interval) wind response in each direction to non-directional N-year MRI is ascertained by Laboy, et al. (2013). It has been reported by Ellingwood, M.ASCE and Tekie (1999) that K_d is considered as one of the factors in lowering the wind pressure coefficient through BLWTL (boundary layer wind tunnel laboratory test) conducted. In addition, Laboy, et al. (2013) also mentioned that K_d has a consequential impact on wind response such as wind load and pressure. Wind directionality factor advantages structure of buildings and hence, it is vital to determine consensus on wind directionality factor as reduction factor (Isyumov, et al., 2014).

Through the study carried out by Irwin, Garber and Ho (2005), explicit consent for the directionality factor to be applied in the evaluation of wind responses has not been clarified. According to Hughes (2015), a non-directional approach is considered to have a more critical case than the directionality of either twelve sectors (each 30°) or four quadrants (each 90°). Chock, Peterka and Yu (2005) mentioned that lower possibility is determined for the aerodynamical worst-case and critical wind direction acting on building structures. Moreover, the speed-up effect due to topographic conditions has a considerable effect on the K_d value in which the scenario is neglected in ASCE standards (Chock, et al., 2005). A probabilistic analysis conducted by Vega-Avila (2008) in determining the variation coefficients and exceedance probability of K_d has engendered the suggestion of disregarding the wind directionality factor in design codes or standards.

2.10.8 Seasonal Variation

Due to changes in the season with distinct weather and temperature, wind speed and wind direction have been influenced. Seasonal factor (S_s) is introduced by British Standard Institution (BSI) (2002) as a reduction factor for wind speed, and it is considered in the design for buildings that experience temporary works. S_s is necessary to be taken into account for buildings under construction. Value of 1 for S_s is applied for permanent buildings that experience wind flow continuously for a period of time exceeding 5 months (British Standard Institution (BSI), 2002). Besides, seasonal variation is determined to be more vulnerable compared to directional variation due to the independence and absolute of its occurrence (Cook, 1983). Pressure difference due to the seasonal variation is verified by Tarabath (2010), and airflow induced by the seasonal variation is termed as the seasonal wind. As stated by Digest (1989), the seasonal factor with a lower partial safety factor is determined for the building structure, which is only exposed to one season. Besides, Cook (1983) stated that consideration of seasonal factors for temporary building structures is distinguished among two various types of short-duration exposure, which are for the period within a year and a short period every year. Through findings by Wu, Mok and Cheng (2011), the variation of the season has a significant effect on gust factor in which higher gust factor with the value of more than two is recorded during summertime. Likewise, seasonal variation is also correlated to temperature variation, as determined by Aachen (2005), and the consideration of thermal action on buildings is significant to secure the functionality and safety of buildings. The significant external temperature, which comprises of the function of the orientation of building and characteristics of thermal absorption, is applied for summer and wintertime (Vrouwenvelder and Steenbergen, 2005).

2.10.9 Gust Factor

According to Ranjitha, et al. (2014), the gust factor is defined as the proportion of peak wind speed to average wind speed for a while. Besides, as studied by Hamzah, Usman and Omar (2018), the gust response factor acts as a multiplier for the conversion design wind action to extreme wind action. Significance of gust response factor considered for design wind load is affirmed due to its important characteristics of resulting substantial wind speed variation, magnifying amplitude, and altering wind orientation (Kwon and Kareem, 2009). As claimed by Kwon and Kareem (2013), consideration of gust factor by various standards due to its effect on a tall building with the existence of discrepancies in wind parameters termed.

According to Wang, Hu and Cheng (2011), reduction of gust factor following mean wind speed, which is larger than the critical value under the state of flat upstream terrain, is determined. Moreover, for several typhoons studied, different correlations between gust factor and turbulence intensity are obtained. The reduction of the gust factor is determined when a more extended averaging period is encountered, as shown in Figure 2.23 (Cao, et al., 2009).

Terrain conditions, the roughness of the terrain, and the dimension of the structure have a significant effect on gust factor. Gust impact on buildings is reduced when a higher dimension of buildings is encountered, which upholds the phenomenon of spatial averaging. Besides, the magnitude of gust factor is reported to appear lower in the area situated in offshore and uncovered higher level, while higher in places crowded with buildings and occupied with compound topographic features (Wu, et al., 2011). Hence, it is verified that turbulence intensity is related to gust factor as claimed by Ghanadi, et al. (2017) because the increment of roughness coefficient of a surface has a positive effect on gust factor within the atmospheric boundary layer. Urban terrain with a higher roughness coefficient resulting in higher wind gust speed than smooth terrain in the rural area by 50% is determined through research conducted by Ghanadi, et al. (2017).

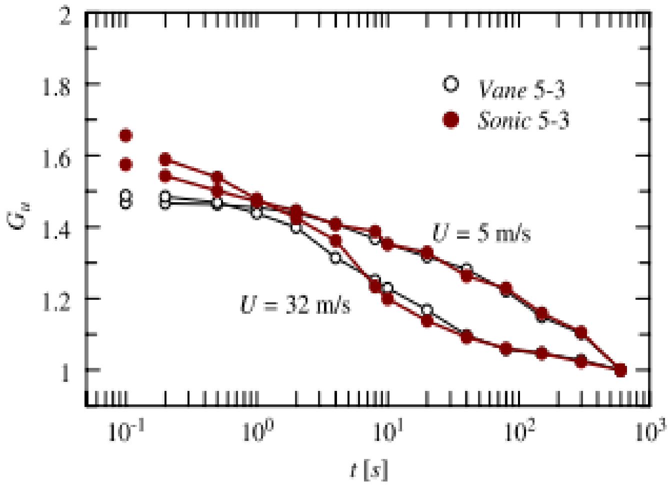

Figure 2.23: Variation of Gust Factor with Gust Averaging Period (Cao, et al., 2009).

2.11 Comparison between Codes

Codes of EN 1991-1-4:2004, BS 6399-2:1997 and AS/NZS 1170.2:2011 are reviewed and compared. A comparison of codes is necessary to recognise the accountability of parameters or factors assumed and applied to the calculation to assure conservative wind load design for buildings in each localised region. The variables analysed and compared are basic wind velocity, wind velocity factors, external and internal pressure on buildings.

2.11.1 Basic Wind Speed

Basic wind speed is an important parameter considered to obtain regional wind speed for the calculation of wind load and pressure. Each standard has different basic wind speed defined with the meantime and returns period. Table 2.4 below has indicated the definition of essential wind speed for several standards, including EN 1991-1-4:2004 and AS/NZS 1170.2:2011. BS 6399-2:1997 has an average time of 1 hour and a return period of 50 years (British Standard Institution (BSI), 2002). Regional or reference wind speeds are not suggested in EN 1991-1-4:2004; however, they are provided in the discrete National Annex for various countries. In Malaysia, essential wind speed is divided into two zones in Peninsular Malaysia, as stated in MS 1553: 2002 (Department of Standards Malaysia, 2007). According to Holmes (2001), the application of hourly wind speed is appropriate to obtain results for topographic factors in BS 6399-2:1997. In AS/NZS 1170.2:2011, different regions for cyclonic or non-cyclonic zones are distributed in the map of Australia and New Zealand for the selection of reference wind speed. Additionally, regional wind speed is also applicable for many average recurrence years or return period, R larger than five years (Joint Standards Australia, 2011).

Table 2.4: Definitions of Basic Wind Speed (Holmes, 2001).

Code	Averaging time	Basic return periods
ISO 4353:2009	3 s (10 min)	Not specified
EN 1991-1-4.6	10 min	50 years
ASCE 7-10	3 s	350-700-1700 years
AIJ	10 min	100 years
AS/NZS 1 170.2:2011	0.2 s	500-1000 years

2.11.2 Wind Speed Factors

In EN 1991-1-4:2004, wind speed factor of roughness factor, C_r and orography factor, C_o are used to obtain average wind speed, v_m for extreme wind-pressure, q_p. Subsequently, q_p is continued for the computation of external wind pressure, w_e. According to CEN (2004), C_r considers the effects of the height above ground level and roughness length of terrain. Categories and parameters of terrain are defined in EN 1991-1-4:2004 and AS/NZS 1170.2:2011 based on distribution of logarithm velocity (Holmes, 2001). C_o is considered when effect of larger than 5% is determined on wind speed and the value is further attained from national annex (CEN, 2004). On the other hand, wind speed computed in BS 6399-2:1997 takes into account several components like S_a, S_d, S_s and S_p (British Standard Institution (BSI), 2002). S_a examines the effect due to various height above sea level on wind velocity while S_d reviews the wind velocity under the influence of wind direction (angle intervals). As stated by Holmes (2001), S_d value manages to reduce wind action for buildings subjected to wind response within a year. According to Joint Standards Australia (2011), a particular shielding effect of buildings is considered for wind speed in AS/NZS 1170.2:2011 despite the effect due to wind direction, terrain condition, and site topographic feature. Distribution of shielding multiplier, M_s according to shielding parameter, s is presented in Table 2.5. On top of that, shielding parameter is obtained through formula with takes into consideration the dimension of buildings and shielding space (Joint Standards Australia, 2011).

Table 2.5: Shielding Multiplier (Joint Standards Australia, 2011).

Shielding parameter (s)	Shielding multiplier (M_s)
≤1.5	0.7
3.0	0.8
6.0	0.9
≥12.0	1.0

2.11.3 External Pressure on Buildings

On the outer surface of buildings, wind pressure acting on the roofs, claddings, and vertical walls of buildings are computed. In EN 1991-1-4:2004, external pressure coefficient, C_{pe} is required in the computation for wind pressure and tabulated data of C_{pe} values are presented in Table 2.1. C_{pe} is distributed into the loading area of 1 m^2 ($C_{pe,1}$) and 10 m^2 ($C_{pe,10}$) to anticipate the critical effect of pressure exerted on smaller compartments (CEN, 2004). Besides, interpolation is allowable for loading area within the range from 1 to 10 m^2

(Holmes, 2001). As mentioned by British Standard Institution (BSI) (2002), size effect factor, C_a is considered by examining the diagonal dimension of building surface despite C_{pe} value. On top of that, C_a is obtained depending on the effective height and terrain category. The standard method of directional approach is applied for pressure coefficient in BS 6399-2:1997 for every wind direction with the increment of 15°. For AS/NZS 1170.2:2011, aerodynamic shape factor, C_{fig} is included in the computation of external pressure on building compartments. In order to compute C_{fig}, external pressure coefficient, local pressure factors, K_l and other factors are determined. These parameters are obtained from each category of tabulated data comprised the functions of the aspect and span ratio of buildings (Joint Standards Australia, 2011).

2.11.4 Internal Pressure on Building

In opposition to external pressure, the computation of internal pressure acting inward on building compartments is also significantly important. In EN 1991-1-4:2004, internal pressure coefficient, C_{pi} is determined by factorising the C_{pe} under the condition for internal dominant building face. For the buildings with openings equally allocated, C_{pi} which comprises the value from -0.5 to +0.35 is obtained from the graph with h/d ratio and opening ratio, μ is taken into consideration as illustrated in Figure 2.24 (CEN, 2004). In BS 6399-2:1997, a uniform positive and negative value of C_{pi} is assumed for enclosed buildings with the consideration of permeability of building walls. While for buildings with dominant openings, factorisation of C_{pe} is applied as in EN 1991-1-4:2004. Additionally, BS 6399-2:1997 also anticipates C_{pi} value for open sided building which is determined by considering wind flow direction and number of open faces as revealed in Table 2.6 (British Standard Institution (BSI), 2002). According to Holmes (2001), two tables are used to decide the positive and negative value of C_{pi} in AS/NZS 1170.2:2011. Selection of C_{pi} value is based on the different permeability of walls, ratio of opening area in dominant face to sum of opening area and location of dominant opening (Joint Standards Australia, 2011).

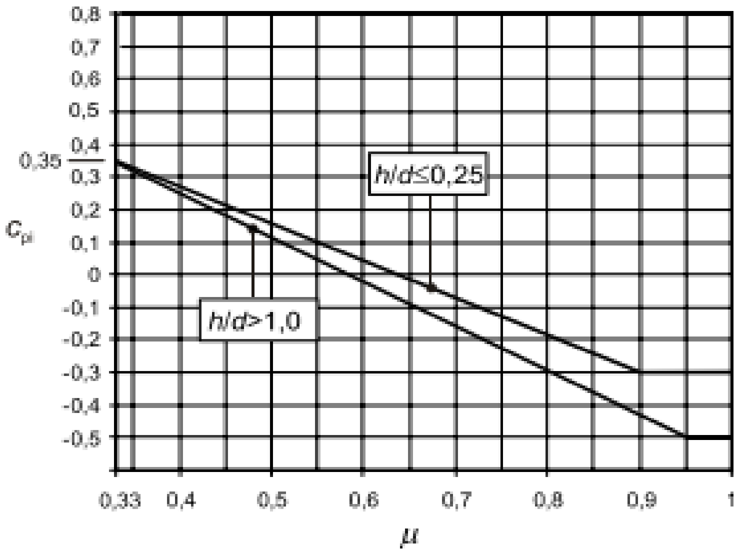

Figure 2.24: Internal Pressure Coefficients for Uniformly Distributed Openings (CEN, 2004).

Table 2.6: Internal Pressure Coefficient for Open-sided Buildings (British Standard Institution (BSI), 2002).

Wind direction θ	One open face		Two adjustment open faces	Three open faces
	Shorter	Longer		
0°	+ 0.85	+ 0.80	+ 0.77	+ 0.60
90°	- 0.60 + 0.52	- 0.46 + 0.67	- 0.57 + 0.77	- 0.63 + 0.40
180°	- 0.39	- 0.43	- 0.60	- 0.56

2.12 Summary

Past studies on the wind load effects on high rise buildings were reviewed. The wind load was explained and defined clearly. Next, various wind-related disasters were reviewed. Besides, the topic of along and across-wind loading was also reviewed. Next, high-rise building was defined, and parameters related to height in different codes were reviewed. Moreover, the topic related to wind speed variation on tall buildings was discussed. Subsequently, the topic of wind drift and drift control were reviewed. Furthermore, wind pattern in Malaysia was assessed in this chapter. Additionally, several parameters of wind effects were evaluated. The parameters such as aerodynamic modifications of buildings, the shape of buildings, aspect ratio of buildings, interference effect, topographic effect, terrain condition, wind directionality, seasonal variation, and gust factor were discussed. Ultimately, a comparison between codes was made on the topics of basic wind speed, wind speed factors, external pressure, and internal pressure on buildings. Through these literature reviews of past researches and studies, it is discovered that topics based on the comparison of wind load for various wind design codes were in scarcity. This project report is carried out with the hope to contribute to the research gap, as well as aid in consideration of wind load effect on Malaysia’s building structure.

CHAPTER 3

METHODOLOGY AND WORK PLAN

3.1 Introduction

In this research project, this chapter focuses on the workflow and essential criteria to be considered. Relevant issues and subjects involved in the execution of the research are explained and examined. Chapter 3 begins with the illustration of a flowchart of work for the project. Secondly, wind codes chosen for wind load design are specified after the parametric study has been performed. Next, structural analysis software is introduced, and some applications of SCIA Engineer in the structural analysis are clarified. Then, two high-rise buildings are modelled with properties and dimensions of concrete material, structural components, and steel reinforcement specified. Subsequently, the assignation of permanent and imposed loads on the building model is clarified. The analysis result for the project is also discussed. Ultimately, a summary of chapter 3 is done.

3.2 Flowchart of Work

During this study, the methodology was started with a parametric study based on Eurocode (EC), British Standard (BS), and Australia/New Zealand (AS/NZS) codes. Next, the high-rise building models were developed by software analysis compliance with the scope and limitation of the study. Afterwards, permanent loads and imposed loads were assigned to the building models. Besides, wind loading was also assigned with compliance to EC standard wind code. Subsequently, linear analysis is performed on the building model through structural software utilised. Finally, the relevant results of the horizontal deflection of the building model were presented and compared. Figure 3.1 below shows the flowchart of work for this project.

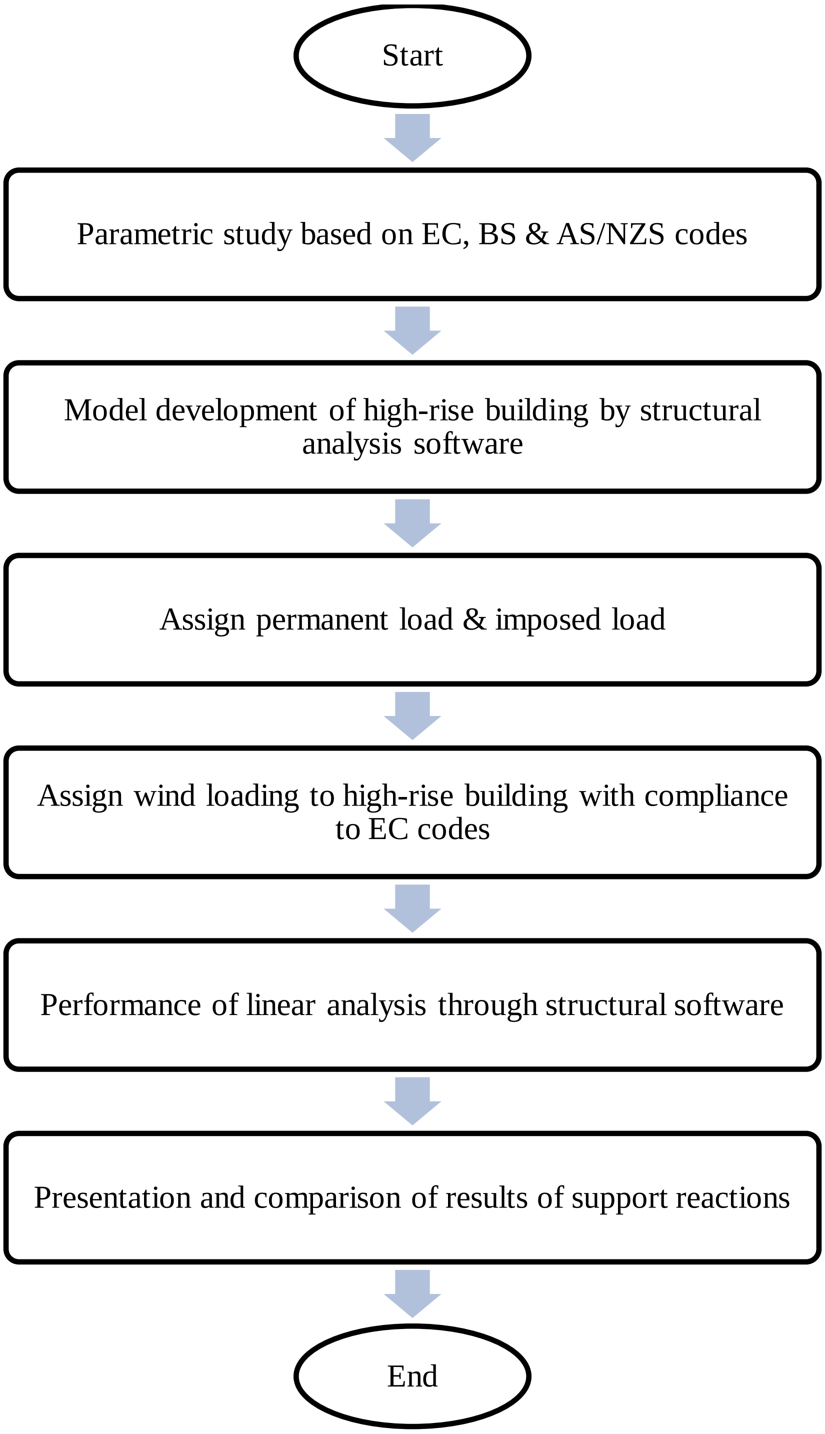

Figure 3.1: Flowchart of Work.

3.3 Parametric Study

3.3.1 Wind Load based on European Standard

In the determination of the design load due to wind actions, the computation of wind pressure was examined. According to CEN (2004), the external wind pressure is computed using the Equation (3.1). The external pressure coefficient is obtained from Table 7.1 in EN1991-1-4:2004, clause 7.2.2.

$$w_e = q_p(z_e)c_{pe} \tag{3.1}$$

where:

$q_p(z_e)$ = extreme speed pressure, Pa

z_e = reference height of external pressure

c_{pe} = pressure coefficient of external surface

After wind pressure is attained, wind force acting on structural components can be derived directly from surface wind pressure with Equation (3.2). Meanwhile, the direct wind force for the building, which is divided into parts with different wind profiles distributed, is computed with Equation (3.3) for structural components. The value of structural factor considers the effect of building height and is referred to in EN 1991-1-4:2004, clause 6.2. However, the force coefficient is obtained by expression in EN 1991-1-4:2004, clause 7.6.

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{3.2}$$

$$F_w = c_s c_d \cdot \sum_{elements} c_f \cdot q_p(z_e) \cdot A_{ref} \tag{3.3}$$

where:

$c_s c_d$ = structural factor

c_f = force coefficient of structure or structural component

A_{ref} = reference area of a structure or structural component, m^2

3.3.2 Wind Load based on British Standard

On the other hand, wind pressure exerting on the external building's surface, p_e is computed with Equation (3.4), as stated by British Standard Institution (BSI) (2002). C_{pe} value is obtained from Table 5 in BS6399-2:1997, clause 2.4.2 while C_a value is referred in clause 2.1.3.4.

$$p_e = q_s C_{pe} C_a \tag{3.4}$$

where:

q_s = dynamic pressure, Pa

C_{pe}= pressure coefficient of external surface

C_a = size effect factor for external pressure

To obtain wind load, P based on BS 6399-2:1997, Equation (3.5), Equation (3.6) and Equation (3.7) are applied, which can be distributed for enclosed buildings and components of the building. In detail, C_p value for the overall structure is obtained from Table 5a, as presented in BS 6399-2:1997, clause 2.4.2, while for structural elements, it is referred to in BS 6399-2:1997, clause 2.7 (British Standard Institution (BSI), 2002).

$$P = pA \tag{3.5}$$

For enclosed building:

$$p = p_e - p_i \tag{3.6}$$

For building components:

$$p = q_e C_p \tag{3.7}$$

where:

p = net wind pressure, Pa

A = area of loading, m^2

p_i = internal wind pressure, Pa

q_e = dynamic pressure (effective wind speed), Pa

C_p = net pressure coefficient

3.3.3 Wind Load based on Australia & New Zealand Standard

As mentioned by Joint Standards Australia (2011), the design wind pressure is computed based on Equation (3.8). As specified in AS/NZS 1170.2:2011, clause 2.3, $V_{des,0}$ is assumed as $V_{sit,\beta}$ which is described in Section 3.8.3. C_{fig} value is reached in AS/NZS 1170.2:2011, clause 5.2, which considers several factors. Besides, C_{dyn} is determined based on the fundamental frequency of the structure, which can be taken as 1.0 or obtained from clause 6.2 for along wind direction and 6.3 for crosswind direction (Joint Standards Australia, 2011). These parameters are also available in MS 1533:2002.

$$p = (0.5\rho_{air})\left(V_{des,0}\right)^2 C_{fig} C_{dyn} \quad (3.8)$$

where:

p = design wind pressure, Pa

ρ_{air}= air density which assumed as 1.2 $\mathrm{kg/m^3}$

$V_{des,0}$ = building orthogonal wind velocity, $\mathrm{m/s}$

C_{fig} = aerodynamic shape parameter

C_{dyn} = dynamic response parameter

After the determination of the design wind pressure, the force due to wind pressure is computed based on Equation (3.9) (Joint Standards Australia, 2011).

$$F = \sum(p_z A_z) \quad (3.9)$$

where:

p_z = design wind pressure at height z, Pa

A_z = loading area, $\mathrm{m^2}$

3.4 Important Parameters of Wind Codes

Various parameters or factors applied in design wind pressure and wind velocity were determined based on different sections described in EN 1991-1-4:2004, BS 6399-2:1997 and AS/NZS 1170.2:2011. Understanding of tables or expressions utilised for the wind load parameters was compulsory to evaluate the differences between the three wind codes.

3.4.1 More Parameters in European Standard

External wind pressure, w_e is computed with peak pressure, $q_p(Z)$ acting as essential elements. Turbulence intensity, $I_v(z)$ is obtained through Equation (3.10) and Equation (3.11) with concerning a certain height. As stated in EN 1991-1-4, the following equations are used to obtain q_p and q_b for certain reference height, Z_e (CEN, 2004).

$$q_p(Z) = 0.5[1 + 7I_v(z)](\rho_{air})(v_m(z))^2 = c_e(z)q_b \quad (3.10)$$

$$q_b = 0.5\rho_{air} \cdot {v_b}^2 \quad (3.11)$$

where:

$I_v(z)$ = turbulence intensity

$v_m(z)$ = mean wind speed, $\mathrm{m/s}$

$c_e(z)$ = exposure factor

q_b = basic wind pressure, Pa

Furthermore, $V_m(z)$ at a specific height, z is obtained through Equation (3.12) that considers the effect of orography and terrain surface (CEN, 2004). In detail, the orography factor is determined through an expression considering the effect of upwind slope and type of orography as stated in EN 1991-1-4, Annex A.3. Besides, the roughness parameter is obtained by applying expression which takes into account the roughness length and type of terrain in EN 1991-1-4, clause 4.3.2.

$$v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b \quad (3.12)$$

where:

$c_r(z)$ = orography coefficient

$c_o(z)$ = roughness coefficient

v_b = basic wind speed, $\mathrm{m/s}$

In addition, v_b considered in EN 1991-1-4 is the local wind speed taken into account the seasonal and directional effect of wind actions, which are determined in MS 1553:2002. The directional and seasonal factor is assumed as 1 for conservative wind speed design or referred to in MS 1553:2002. Besides, initial wind speed is also referred to MS 1553:2002. Equation (3.13) is applied to obtain v_b (CEN, 2004).

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \quad (3.13)$$

where:

c_{dir} = directional factor

c_{season} = seasonal factor

$v_{b,0}$ = initial wind speed, $\mathrm{m/s}$

3.4.2 More Parameters in British Standard

An important parameter of site wind speed, v_s is determined by Equation (3.14), which comprises of several factors taken into consideration. Basic wind speed is also referred to in MS 1553:2002. S_a value is obtained through expression in BS 6399-2:1997, clause 2.2.2.2. S_d value is determined from BS 6399-2:1997 Table 3, clause 2.2.2.3. Next, S_s is assumed as 1.0 for permanent building experienced long-period wind exposure. Then, S_p is also taken as 1.0 for the general design of the building.

$$v_s = v_b \times S_a \times S_d \times S_s \times S_p \tag{3.14}$$

where:

v_b = basic wind speed

S_a = altitude component

S_d = direction component

S_s = seasonal component

S_p = probability component

Despite site wind speed, effective wind speed is also determined in British Standard, which takes into account the direction of wind action for structure and type of site. Equation (3.15) is used to compute the effective wind speed.

$$v_e = v_s \times S_b \tag{3.15}$$

where:

v_e = effective wind speed, m/s

S_b = terrain & building factor

For terrain & building factor, sites in country and town terrain are distributed and with Equation (3.16) and Equation (3.17), respectively (British Standard Institution (BSI), 2002).

For country region:

$$S_b = S_c\{1 + (g_t \times s_t) + S_h\} \tag{3.16}$$

For town region:

$$S_b = S_c T_c\{1 + (g_t \times s_t \times T_t) + S_h\} \tag{3.17}$$

where:

S_c = fetch factor

S_t = turbulence factor

g_t = gust peak factor

S_h = topographic increment

T_c = fetch adjustment factor

T_t = turbulence adjustment factor

Fetch and turbulence factors for the county and town terrain are obtained in BS 6399:1997 Table 22 and Table 23, clause 3.2.3.2 and 3.2.3.3, respectively. The determination of the value for S_h is referred to in BS 6399-2:1997, clause 3.2.3.4. Moreover, g_t is determined from BS 6399-2:1997 Table 24, clause 3.2.3.3.

3.4.3 More Parameters in Australia & New Zealand Standard

Parameter of site wind speed, $v_{sit,\beta}$ is affected by several multipliers, as shown in Equation (3.18) (Joint Standards Australia, 2011).

$$v_{sit,\beta} = v_R M_d\left(M_{z,cat} M_s M_t\right) \tag{3.18}$$

where:

v_R = regional 3s gust wind speed, m/s

M_d = wind directional factor

$M_{z,cat}$ = terrain/height factor

M_s = shielding factor

M_t = topographic factor

National Annex is applied to attain localised parameters in Malaysia. According to MS 1553:2002 Table 3.1, clause 3.2, v_R is obtained depending on locations selected with a distinct recurrence interval of 20, 50, and 100 years. Furthermore, M_d is assumed as 1.0. Next, $M_{z,cat}$ is obtained in clause 4.2.2, with values indicated in MS 1553:2002 Table 4.1. In detail, four terrain categories with distinct characteristics are described in the section. Moreover, M_s is referred to in MS 1553:2002 Table 4.3, clause 4.3.1. Finally, M_t is taken as hill shape multiplier, M_h according to MS 1553:2002, clause 4.4, with each height of topography features and horizontal upwind distance distinguished (Department of Standards Malaysia, 2007).

3.5 Structural Analysis Software

Civil engineering software provides users with assistance in managing construction costs, scheduling, the arrangement of material resources, and the design of building structures. Structural analysis and design software are widely used to ease up the work and enhance the efficiency or productivity of engineers. In this project, a functional and practical software like SCIA Engineer introduced in the marketplace is utilised.

To achieve the goals of this project, SCIA Engineer 19.1 is used to analyse building and structural response due to wind load following three different standard codes. SCIA Engineer is a software founded in 1974 which is operated by Nemetschek Group. It is capable of providing concrete design, steel design, composite structure design, meshing & analysis, loading analysis, BIM, and modelling. According to Dubey (2017), SCIA stands for boosted productivity, speed, transparency, precision, and economical design. The general applicability of SCIA Engineer software advantages civil engineers by contributing to their goals of building an either well organised or cost-effective construction project.

According to SCIA Structural Design and Analysis Software (2020), generation and application of 3D wind loads on building structures can be carried out complying with provisions as stated in codes such as Australian/New Zealand Standard, British Standard, and European Standard. More straightforward evaluation and demonstration of wind pressure coefficient and wind loads applied to various locations are attained through the application of SCIA Engineer with its excellent performance of graphical interface, as shown in Figure 3.2.

Figure 3.2: Graphical View of 3D Wind Generator (SCIA Structural Design and Analysis Software, 2020).

3.6 Modelling of High-rise Building

A High-rise building model was generated before structural analysis was carried out with SCIA Engineer 19.1. The dimension of the building was defined for the categorisation of the building as slender or tall building through a parametric study based on EN 1991-1-4:2004, BS 6399-2:1997 and AS/NZS 1170.2:2011. In this study, there are two building models developed in obtaining the data of building response for comparative analysis. For the first model, a rectangular-shaped high-rise building modelled was defined with eight floors. In particular, the floor height of the building was set to 3 m, and the total height of the building modelled was 24 m, excluding the height of the stump. Besides, span ratio (h/d) of 1:5 and aspect ratio (h/b) of 1:2.5 were specified for the building in deciding the alongwind and crosswind width of the building model. The structure of the building first modelled was presented in Figure 3.3, and the building layout for the single building model is shown in Figure 3.4.

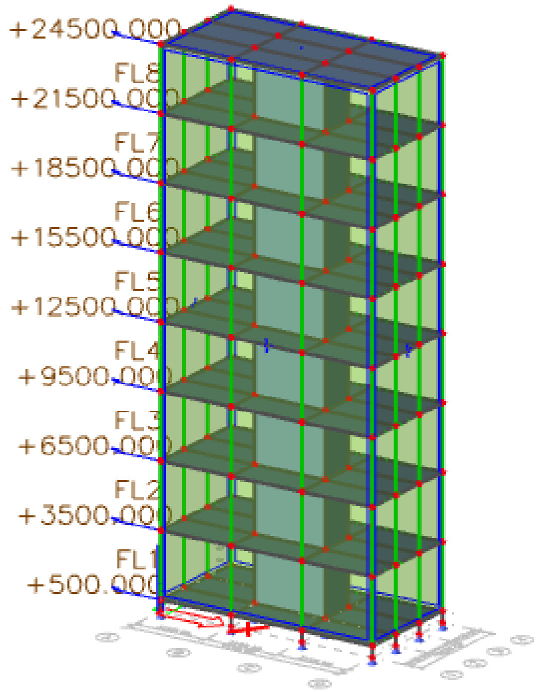

Figure 3.3: Structure of first building model.

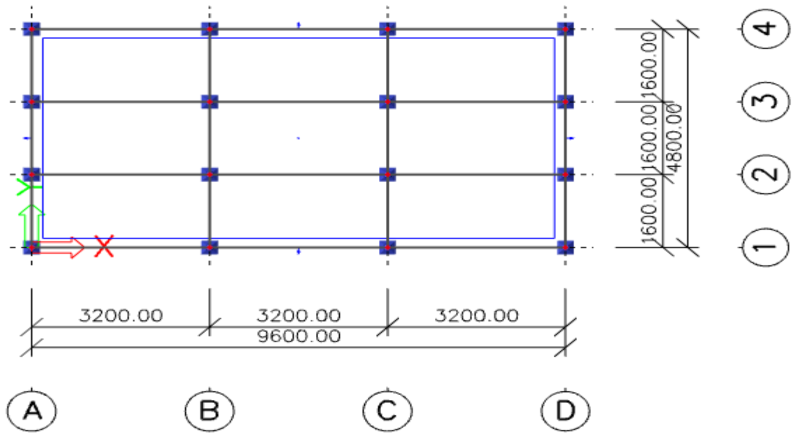

Figure 3.4: Building layout for single building model.

During the modelling of the tall building, structural components of beam, slab, column and shear wall were included. Appropriate dimensions of each element were decided for the building model, as presented in Table 3.1. The columns were allocated at the outer structure of the building, while the shear wall was allocated as inner vertical components for the structure. Beams connected to vertical components were allocated on every floor, and the ground beam was also included at the ground level of the building model. The slabs were allocated on top of beams for every floor, and the roof slab was allocated on top of the model. For the building model, 500 mm was decided for the height of the stump for the connection joints between columns and foundation. Moreover, the fixed foundation support of the building was decided. Reinforced concrete was the material chosen for the building model with a grade of C30/37. Meanwhile, the steel reinforcement of B 500A was chosen in the study. In detail, the properties of concrete and steel reinforcement were clarified, as shown in Table 3.2 and 3.3.

Table 3.1: Structural Components Description.

Structural Components	Description
Column dimension	$1000\text{ mm} \times 600\text{ mm}$
Beam dimension	$600\text{ mm} \times 400\text{ mm}$
Slab thickness	200 mm
Wall thickness	200 mm

Table 3.2: Concrete Properties.

Properties	Unit
Concrete unit mass	2500 kg/m^3
Elastic Modulus, E	3.280E4 MPa
Shear Modulus, G	1.367E4 MPa
Compressive strength, f_{ck}	30 MPa

Table 3.3: Steel Reinforcement Properties.

Properties	Unit
Steel unit mass	7850 kg/m^3
Elastic Modulus, E	2.000E5 MPa
Shear Modulus, G	8.333E4 MPa
Yield strength, f_{yk}	500 MPa
Maximum tensile strength, f_{tk}	525 MPa

For the second model, a twin rectangular-shaped high-rise building sitting on a podium was modelled. Like the first model, the high-rise building was defined with the same number of floors, floor height, span ratio, and aspect ratio. For the podium of the second model, 5 floors were defined with a floor height of 3 m. Therefore, a total height for the second model was 39 m, excluding the height of the stump. The dimension of the podium was decided to be 32000 mm x 8000 mm. The structure of the building second modelled was presented in Figure 3.5. Figure 3.6 shows the dimension of the building structure is indicated through the building layout.

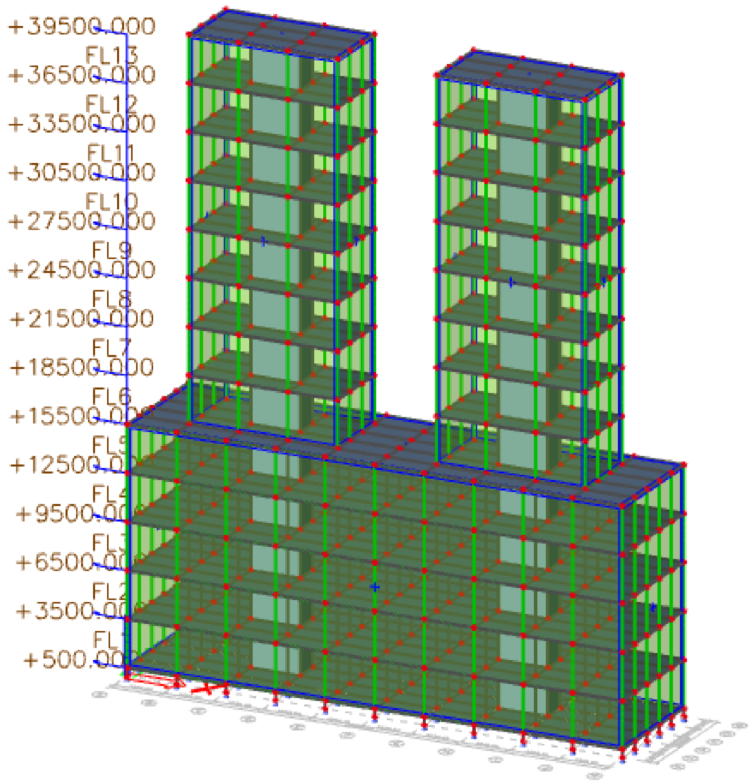

Figure 3.5: Structure of second building model.

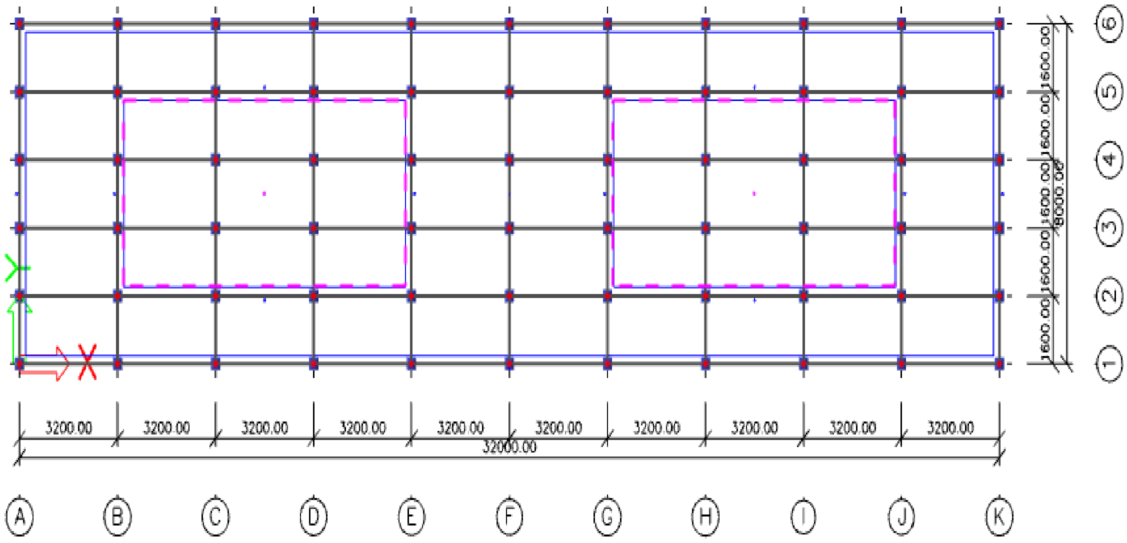

Figure 3.6: Building layout of twin building model with podium.

Like the first model, the second model also included structural components of the beam, slab, column, and shear wall with the same dimension. Equal allocation of structural components was decided for the second model. Stump with the same height of 500 mm was also decided for the connection joints between columns and foundation. Then, the hinged foundation support of the building was also decided. Materials chosen for reinforced concrete and steel reinforcement were the same as for the first model.

3.7 Permanent Load & Live Load

Despite the assignation of wind load on the building model, structural loading was mainly taken into account during building design. Permanent load and imposed load were additional loads generally considered for buildings. Permanent loads due to the construction of brick wall and application of floor finishing were also considered as significant loads to be sustained by building models instead of self-weight of the building itself. As shown in Table 3.4, permanent loads of structural elements that were obtained from unit weights were assumed and considered during the assignation of loading. Floor finishes of 50 mm were considered. Besides, a standard brick which has a loading of 2.6 kN/m^2 was multiplied with a floor height of 3m, which was assumed during the modelling of the high-rise building.

Table 3.4: Permanent Loads of Structural Components.

Structural Components	g_k (kN/m)	g_k (kN/m^2)
Floor finishing	-	1.20
Brick wall	7.8	-

In this study, the building model analysed was considered as a condominium with most of the area of the building used explicitly for domestic and residential activities. According to MS EN 1991-1-1:2010, categorisation of various potential imposed live loads could be determined. In this study, for the first model, only imposed loads for the domestic and residential area was applied. While, for the second model, imposed loads of traffic area was considered and allocated on the podium. The potential imposed loads on the building models were selected and presented in Table 3.5 below.

Table 3.5: Imposed Loads on Building Model (Department of Standards Malaysia, 2010).

Specific Use	Category	q_k (kN/m^2)
Area for domestic & residential activities	A	2.5
Traffic Area (gross vehicle weight ≤ 30kN)	F	2.5

3.8 Load Combination

To determine the ultimate condition of building response, load combination is necessary to be applied. According to MS EN 1990-2002 cl. 6.4.3.2(3), the combination of actions for persistent or transient design situation is used. In this study, the ultimate design load is determined using Equation (3.19) with compliance of all the clauses stated in the standard code.

$$1.35G_k + 1.5Q_{k,1} + 0.75Q_{k,w} \qquad (3.19)$$

where:

G_k = permanent load

$Q_{k,1}$ = live load

$Q_{k,w}$ = wind load

3.9 Analysis of Results

Besides, after the assignation of loading and appropriate parameters or factors, a linear analysis was performed on both tall building models. The results of the analysis obtained for both models were extracted and compared. In the comparative study, the relevant results of support reactions were considered. The percentage differences of support reactions on columns and shear walls were determined. Potential factors contributing to the difference in reactions were determined and found.

3.10 Summary

In conclusion, the overall description of the flow of works carried out in this project was discussed. Expressions of wind load in three codes: EN 1991-1-4:2004, BS 6399-2:1997 and AS/NZS 1170.2:2011 were indicated. Parameters or factors of wind codes were also discussed. The modelling of building structures is performed, and relevant requirements or specifications of loading are explained. Lastly, the analysis results of building models for comparative study are discussed.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Standard Code Evaluated

The standard code used in the evaluation of the result of building response due to wind load is European Standard (EN 1991-1-4:2004). Several factors which lead to the chosen of European Standard are explained. The coefficient related to size factor and dynamic factor, which takes into account peak factor and turbulence intensity, are parameters considered by European Standard. European Standard takes into account the structural factor, $c_s c_d$ which fully contemplates the background factor, B and resonance response factor, R of building response. However, instead of considering both size and dynamic factor, Australia & New Zealand Standard only consider dynamic factor. Moreover, more significant velocity pressure and drag force coefficient are considered by European Standard than Australia & New Zealand Standard, and British Standard would result in a more massive wind effect on buildings. With that, higher base shear and base moment with larger safety factor are obtained with the application of Eurocode Standard.

Besides, apart from Australia & New Zealand Standard, European Standard considers the distribution of wind velocity pressure profile along the vertical wall of building into several regions depending on parameters of height, h and crosswind width, b of building. Beyond the windward wall, distribution of wind pressure into various zones is also executed on the sidewall of the building. Also, allocation of wind pressure on the roof of building for European Standard is performed in a more detailed way than Australia & New Zealand Standard. According to European Standard, several roof regions are distinguished depending h and b on a building. Hence, with the various categorisation of regions on building surfaces, more appropriate pressure coefficient is utilised following European Standard. Additionally, two types of pressure coefficient distinguished between the building and small elements are acknowledged.

4.2 Results

Both structure models, single high-rise building and twin high-rise building sitting on the podium, were constructed in this project. The result of support reactions on columns and shear walls were obtained through linear analysis performed using SCIA Engineer. For a building model, load combination under the ultimate limit state (ULS) was considered for the determination and evaluation of support reactions. Besides, for each model, four types of load combination were evaluated in which different grouping of the positive or negative coefficient of external (c_{pe}) and internal pressure (c_{pi}) with the wind flow direction of 90°.

Although results for various load cases of different angles like 0°, 90°, 180°, and 270° were obtained, the wind flow direction of 90° was chosen to study in this project. Among the four wind load directions generated on the structural model, 90° results in most unfavourable building response as the higher and critical value of support reactions are obtained. With wind direction of 90°, a significant load effect is engendered when wind flows along the direction of shorter width (along-width) of building models. Also, 90° was selected as the same critical value of support reactions for the wind load generated for the wind flow direction of 270° but in the opposite direction of building models. Figure 4.1 below shows the wind direction of 90° in which wind load acts in the y-axis on the surface of the single building model.

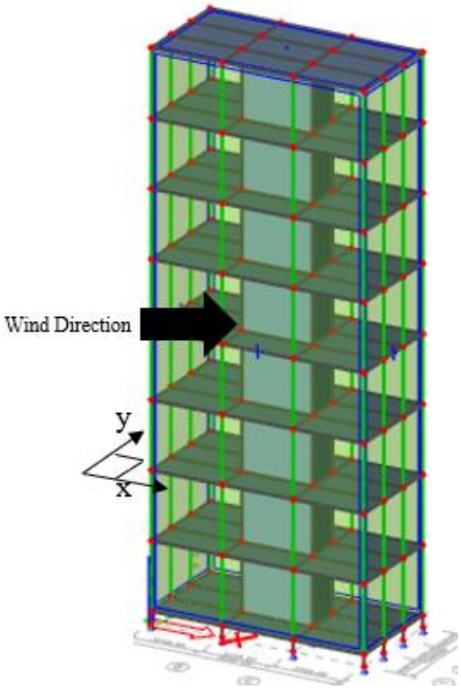

Figure 4.1: Wind load acting at 90° in y-direction on the building.

The different signs for the four combinations of external and internal pressure are illustrated in Figure 4.1 and Figure 4.2 below. As shown in Figure 4.1, when wind flows from left to right direction and openings or voids are located on the windward side, pressure combination of positive external and internal pressure is indicated on the windward side of the building; whereas, pressure combination of negative external and positive internal pressure is indicated on the leeward side of the building. As indicated in Figure 4.2, when wind flows from right to left direction and openings or voids are located on the leeward side, pressure combination of positive external and negative internal pressure is determined on the windward side of the building; whereas, pressure combination of negative external and internal pressure is indicated on the leeward side of the building.

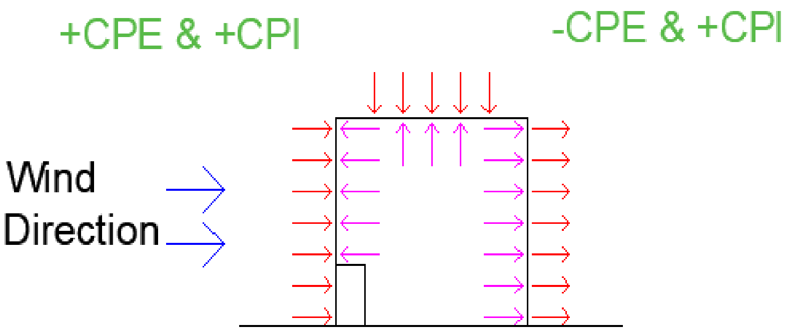

Figure 4.2: External and internal pressure with openings on windward side.

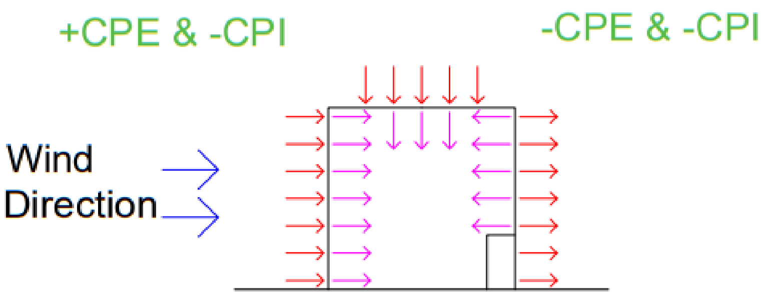

Figure 4.3: External and internal pressure with openings on leeward side.

4.2.1 Support Reactions under ULS of Single High-rise Building Model

The support reactions of each node, as shown in Figure 4.4, are obtained as the results for the single building model studied. For a clear presentation of results, support reactions for four types of load combinations are tabulated together and shown in Table 4.1. From the tabulated data, it is observed that support reactions for columns and shear walls under ULS1 (3DWind with 90°, +CPE & +CPI) and ULS3 (3DWind with 90°, -CPE & +CPI) are identical. The same scenario is also proven for ULS2 (3DWind with 90°, +CPE & -CPI) and ULS4 (3DWind with 90°, -CPE & -CPI). Hence, the building response of reactions due to wind load effect is concluded to have identical values under the scenario with the same sign of internal pressure coefficient. As stated in European Standard, there are two CPI with different signs considered in the computation, which are +0.2 and -0.3. Besides, it can also be concluded that the identical value of external pressure coefficient when along and across-wind flows through the symmetrical building as considered in this building model evaluated.

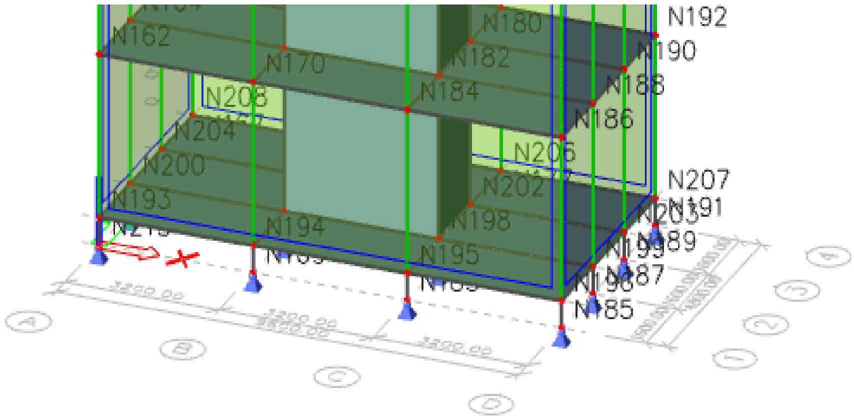

Figure 4.4: Support reactions for columns and shear walls of single building with nodes.

Table 4.1: Support reactions for four types of load combinations on the single building model.

Structure	Node	Rz (kN)			
		ULS 1	ULS 2	ULS 3	ULS 4
COL	Sn16/N213	747.42	749.79	747.42	749.79
COL	Sn1/N163	1073.95	1076.16	1073.95	1076.16
COL	Sn2/N165	1206.08	1208.30	1206.08	1208.30
COL	Sn3/N167	1426.81	1429.18	1426.81	1429.18
COL	Sn4/N169	1014.50	1018.27	1014.50	1018.27
SW	Sn12/N209	635.97	638.25	635.97	638.25
SW	Sn13/N210	1073.01	1075.29	1073.01	1075.29
COL	Sn5/N175	1812.33	1816.10	1812.33	1816.10
COL	Sn7/N183	1017.78	1021.55	1017.78	1021.55
SW	Sn15/N212	637.44	639.72	637.44	639.72
SW	Sn14/N211	1074.48	1076.76	1074.48	1076.76
COL	Sn6/N177	1815.61	1819.38	1815.61	1819.38
COL	Sn8/N185	758.87	761.24	758.87	761.24
COL	Sn9/N187	1086.19	1088.40	1086.19	1088.40
COL	Sn10/N189	1218.33	1220.54	1218.33	1220.54
COL	Sn11/N191	1438.26	1440.63	1438.26	1440.63

4.2.2 Support Reactions under ULS of Twin High-rise Building Model with Podium

The support reactions of each node, as shown in Figure 4.5, are obtained as the results for the model of twin building with podium studied. For a clear presentation of results, support reactions for four types of load combinations are tabulated together and shown in Table 4.2. From the tabulated data, it is observed that values of support reactions obtained for columns and shear walls under ULS1 (3DWind with 90°, +CPE & +CPI) and ULS3 (3DWind with 90°, -CPE & +CPI) are quite close. Besides, the same scenario is also proven for ULS2 (3DWind with 90°, +CPE & -CPI) and ULS4 (3DWind with 90°, -CPE & -CPI). The minor differences between them are due to the minor differences in the support reactions due to wind load.

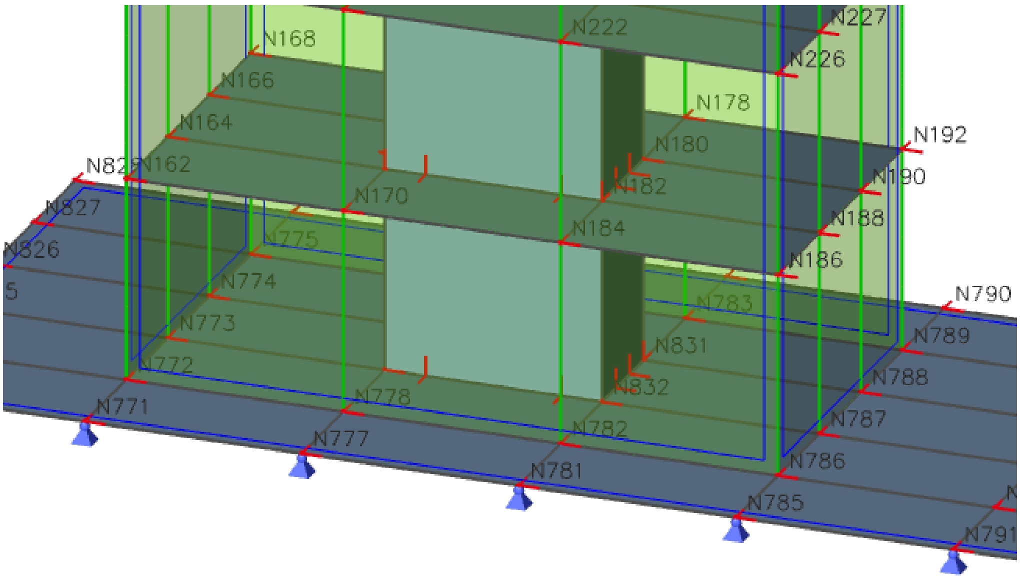

Figure 4.5: Support reactions for columns and shear walls of twin building with podium in nodes.

Table 4.2: Support reactions for four types of load combinations on twin building with podium.

Structure	Node	Rz (kN)			
		ULS 1	ULS 2	ULS 3	ULS 4
COL	Sn68/N772	1149.26	1156.20	1149.47	1156.41
COL	Sn69/N773	1427.93	1434.99	1427.56	1434.62
COL	Sn70/N774	1570.66	1577.73	1569.71	1576.77
COL	Sn71/N775	1788.06	1795.01	1786.51	1793.45
COL	Sn74/N778	1345.64	1354.01	1345.65	1354.03
SW	Sn126/N830	985.45	993.08	984.87	992.50
SW	Sn125/N829	1483.49	1491.12	1482.63	1490.25
COL	Sn75/N779	2072.44	2080.82	2070.46	2078.84
COL	Sn78/N782	1354.19	1362.40	1354.21	1362.41
SW	Sn128/N832	1000.22	1004.60	999.64	1004.02
SW	Sn127/N831	1495.18	1499.56	1494.31	1498.69
COL	Sn79/N783	2075.73	2083.93	2073.74	2081.95
COL	Sn82/N786	1166.47	1173.37	1166.68	1173.58
COL	Sn83/N787	1441.48	1448.60	1441.11	1448.23
COL	Sn84/N788	1581.78	1588.90	1580.82	1587.94
COL	Sn85/N789	1798.78	1805.68	1797.23	1804.12

4.2.3 Support Reactions due to Wind Load for Single High-rise Building Model

For a clear presentation of results, support reactions on the same nodes for the single building, as shown in Figure 4.4 under four types of wind load, are tabulated in Table 4.3. From the tabulated data, it is observed that values of support reactions obtained for columns and shear walls under for wind load with 90°, +CPE & +CPI and 90°, -CPE & +CPI are identical. Besides, the same scenario is also proven for wind load with 90°, +CPE & -CPI and 90°, -CPE & -CPI.

Table 4.3: Support reactions due to wind load.

Structure	Node	Rz (kN)			
		90°, +CPE & +CPI	90°, +CPE & -CPI	90°, -CPE & +CPI	90°, -CPE & -CPI
COL	Sn16/N213	-459.37	-456.21	-459.37	-456.21
COL	Sn1/N163	-94.78	-91.83	-94.78	-91.83
COL	Sn2/N165	81.40	84.36	81.40	84.36
COL	Sn3/N167	446.48	449.64	446.48	449.64
COL	Sn4/N169	-542.25	-537.22	-542.25	-537.22
SW	Sn12/N209	-297.93	-294.89	-297.93	-294.89
SW	Sn13/N210	284.79	287.83	284.79	287.83
COL	Sn5/N175	521.52	526.55	521.52	526.55
COL	Sn7/N183	-542.25	-537.22	-542.25	-537.22
SW	Sn15/N212	-297.93	-294.89	-297.93	-294.89
SW	Sn14/N211	284.79	287.83	284.79	287.83
COL	Sn6/N177	521.52	526.55	521.52	526.55
COL	Sn8/N185	-459.37	-456.21	-459.37	-456.21
COL	Sn9/N187	-94.78	-91.83	-94.78	-91.83
COL	Sn10/N189	81.40	84.36	81.40	84.36
COL	Sn11/N191	446.48	449.64	446.48	449.64

4.2.4 Support Reactions due to Wind Load for Twin High-rise Building Model with Podium

As illustrated in Table 4.4, the support reactions on the same nodes for the twin building with podium as shown in Figure 4.5 for four types of wind load, are tabulated. From the tabulated data, it is observed that values of support reactions obtained for columns and shear walls under for wind load with 90°, +CPE & +CPI and 90°, -CPE & +CPI are deferred by minor difference. The same scenario is also proven for wind load with 90°, +CPE & -CPI and 90°, -CPE & -CPI.

Table 4.4: Support reactions due to wind load.

Structure	Node	Rz (kN)			
		90°, +CPE & +CPI	90°, +CPE & -CPI	90°, -CPE & +CPI	90°, -CPE & -CPI
COL	Sn68/N772	-450.42	-441.16	-450.14	-440.88
COL	Sn69/N773	-113.56	-104.15	-114.06	-104.64
COL	Sn70/N774	76.75	86.17	75.47	84.89
COL	Sn71/N775	401.32	410.58	399.25	408.51
COL	Sn74/N778	-511.49	-500.32	-511.47	-500.30
SW	Sn126/N830	-353.72	-343.55	-354.49	-344.32
SW	Sn125/N829	310.34	320.50	309.19	319.35
COL	Sn75/N779	457.58	468.76	454.94	466.11
COL	Sn78/N782	-503.73	-492.80	-503.72	-492.78
SW	Sn128/N832	-337.31	-331.47	-338.09	-332.24
SW	Sn127/N831	322.63	328.47	321.48	327.31
COL	Sn79/N783	458.31	469.25	455.67	466.61
COL	Sn82/N786	-441.60	-432.40	-441.32	-432.12
COL	Sn83/N787	-110.72	-101.23	-111.22	-101.73
COL	Sn84/N788	76.33	85.83	75.06	84.55
COL	Sn85/N789	401.48	410.68	399.41	408.60

4.3 Support Reactions Comparison

4.3.1 Ultimate Limit State (ULS)

Single building model and twin building model with podium are compared based on their support reactions of columns and shear walls on respective positions under four types of load combination with different types of wind load. The reactions contribute to the large difference with the range approximately from 258.13kN to 422.80kN and the percentage difference between 14% to 57% due to self-weight, permanent load (brick walls & finishing), live load and wind load. However, through data analysis, it is determined that the self-weight of structural components largely contributes to the difference. Due to the extension of the area on the podium floor, more self-weight due to additional beams and slabs are sustained by the columns and shear walls of the building model, which results in a larger reaction. Besides, it is also determined that the reactions of columns due to the live load are lower for the twin building model with podium as the load is sustained by more beams and columns on the podium floor. In contrast, reactions of columns and shear walls due to permanent loads, which are contributed by brick walls and finishing, increase for the twin building model with the podium.

Table 4.5: Comparison of single and twin building with podium under ULS1.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	747.42	1149.26	401.84	54
COL	1073.95	1427.93	353.98	33
COL	1206.08	1570.66	364.58	30
COL	1426.81	1788.06	361.25	25
COL	1014.50	1345.64	331.14	33
SW	635.97	985.45	349.48	55
SW	1073.01	1483.49	410.48	38
COL	1812.33	2072.44	260.11	14
COL	1017.78	1354.19	336.41	33
SW	637.44	1000.22	362.78	57
SW	1074.48	1495.18	420.70	39
COL	1815.61	2075.73	260.12	14
COL	758.87	1166.47	407.60	54
COL	1086.19	1441.48	355.29	33
COL	1218.33	1581.78	363.45	30
COL	1438.26	1798.78	360.52	25

Table 4.6: Comparison of single and twin building with podium under ULS2.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	749.79	1156.20	406.41	54
COL	1076.16	1434.99	358.83	33
COL	1208.30	1577.73	369.43	31
COL	1429.18	1795.01	365.83	26
COL	1018.27	1354.01	335.74	33
SW	638.25	993.08	354.83	56
SW	1075.29	1491.12	415.83	39
COL	1816.10	2080.82	264.72	15
COL	1021.55	1362.40	340.85	33
SW	639.72	1004.60	364.88	57
SW	1076.76	1499.56	422.80	39
COL	1819.38	2083.93	264.55	15
COL	761.24	1173.37	412.13	54
COL	1088.40	1448.60	360.20	33
COL	1220.54	1588.90	368.36	30
COL	1440.63	1805.68	365.05	25

Table 4.7: Comparison of single and twin building with podium under ULS3.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	747.42	1149.47	402.05	54
COL	1073.95	1427.56	353.61	33
COL	1206.08	1569.71	363.63	30
COL	1426.81	1786.51	359.70	25
COL	1014.50	1345.65	331.15	33
SW	635.97	984.87	348.90	55
SW	1073.01	1482.63	409.62	38
COL	1812.33	2070.46	258.13	14
COL	1017.78	1354.21	336.43	33
SW	637.44	999.64	362.20	57
SW	1074.48	1494.31	419.83	39
COL	1815.61	2073.74	258.13	14
COL	758.87	1166.68	407.81	54
COL	1086.19	1441.11	354.92	33
COL	1218.33	1580.82	362.49	30
COL	1438.26	1797.23	358.97	25

Table 4.8: Comparison of single and twin building with podium under ULS4.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	749.79	1156.41	406.62	54
COL	1076.16	1434.62	358.46	33
COL	1208.30	1576.77	368.47	30
COL	1429.18	1793.45	364.27	25
COL	1018.27	1354.03	335.76	33
SW	638.25	992.50	354.25	56
SW	1075.29	1490.25	414.96	39
COL	1816.10	2078.84	262.74	14
COL	1021.55	1362.41	340.86	33
SW	639.72	1004.02	364.30	57
SW	1076.76	1498.69	421.93	39
COL	1819.38	2081.95	262.57	14
COL	761.24	1173.58	412.34	54
COL	1088.40	1448.23	359.83	33
COL	1220.54	1587.94	367.40	30
COL	1440.63	1804.12	363.49	25

4.3.2 Wind Load

Single building model and twin building model with podium are compared based on their support reactions of columns and shear walls on respective nodes due to different types of wind load. The comparative results of both models in terms of percentages differences are indicated in a few tables below. The difference in reaction values from 0.19kN to 66.58kN, with a percentage difference of between 0% to 20% are obtained. From the tabulated data, an increase and decrease of reactions due to wind load are determined on the building model with podium from the single building that can be discovered. The difference in support reactions due to wind action partly affects the difference in reactions of columns and shear walls due to load combination under ULS.

Table 4.9: Comparison of single and podium building for wind load with 90°, +CPE & +CPI.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	-459.37	-450.42	8.95	2
COL	-94.78	-113.56	18.78	20
COL	81.40	76.75	4.65	6
COL	446.48	401.32	45.16	10
COL	-542.25	-511.49	30.76	6
SW	-297.93	-353.72	55.79	19
SW	284.79	310.34	25.55	9
COL	521.52	457.58	63.94	12
COL	-542.25	-503.73	38.52	7
SW	-297.93	-337.31	39.38	13
SW	284.79	322.63	37.84	13
COL	521.52	458.31	63.21	12
COL	-459.37	-441.60	17.77	4
COL	-94.78	-110.72	15.94	17
COL	81.40	76.33	5.07	6
COL	446.48	401.48	45.00	10

Table 4.10: Comparison of single and podium building for wind load with 90°, +CPE & -CPI.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	-456.21	-441.16	15.05	3
COL	-91.83	-104.15	12.32	13
COL	84.36	86.17	1.81	2
COL	449.64	410.58	39.06	9
COL	-537.22	-500.32	36.90	7
SW	-294.89	-343.55	48.66	17
SW	287.83	320.50	32.67	11
COL	526.55	468.76	57.79	11
COL	-537.22	-492.80	44.42	8
SW	-294.89	-331.47	36.58	12
SW	287.83	328.47	40.64	14
COL	526.55	469.25	57.30	11
COL	-456.21	-432.40	23.81	5
COL	-91.83	-101.23	9.40	10
COL	84.36	85.83	1.47	2
COL	449.64	410.68	38.96	9

Table 4.11: Comparison of single and podium building for wind load with 90°, -CPE & +CPI.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	-459.37	-450.14	9.23	2
COL	-94.78	-114.06	19.28	20
COL	81.40	75.47	5.93	7
COL	446.48	399.25	47.23	11
COL	-542.25	-511.47	30.78	6
SW	-297.93	-354.49	56.56	19
SW	284.79	309.19	24.40	9
COL	521.52	454.94	66.58	13
COL	-542.25	-503.72	38.53	7
SW	-297.93	-338.09	40.16	13
SW	284.79	321.48	36.69	13
COL	521.52	455.67	65.85	13
COL	-459.37	-441.32	18.05	4
COL	-94.78	-111.22	16.44	17
COL	81.40	75.06	6.34	8
COL	446.48	399.41	47.07	11

Table 4.12: Comparison of single and podium building for wind load with 90°, -CPE & -CPI.

Structure	Rz (kN)		Difference (kN)	Difference (%)
	Single Building	Podium Building		
COL	-456.21	-440.88	15.33	3
COL	-91.83	-104.64	12.81	14
COL	84.36	84.89	0.53	1
COL	449.64	408.51	41.13	9
COL	-537.22	-500.30	36.92	7
SW	-294.89	-344.32	49.43	17
SW	287.83	319.35	31.52	11
COL	526.55	466.11	60.44	11
COL	-537.22	-492.78	44.44	8
SW	-294.89	-332.24	37.35	13
SW	287.83	327.31	39.48	14
COL	526.55	466.61	59.94	11
COL	-456.21	-432.12	24.09	5
COL	-91.83	-101.73	9.90	11
COL	84.36	84.55	0.19	0
COL	449.64	408.60	41.04	9

4.4 Support Reactions Evaluation

In evaluating the results of support reactions, two support reactions on the side column and shear wall are chosen to compare and study. The side column (N187) on the single building is compared with the side column (N787) on twin building with podium, as shown in both figures below. Besides, as indicated in figures below, shear wall (N211) on the single building is compared with the shear wall (N831) on twin building with podium. In the comparison of results, support reaction due to wind load is selected instead of load combination under ULS. Direct comparison of building response due to wind action is executed as a large percentage of difference is determined for the case of load combination under

ULS, which tends to complexify the evaluation as more factors and aspects are required to consider.

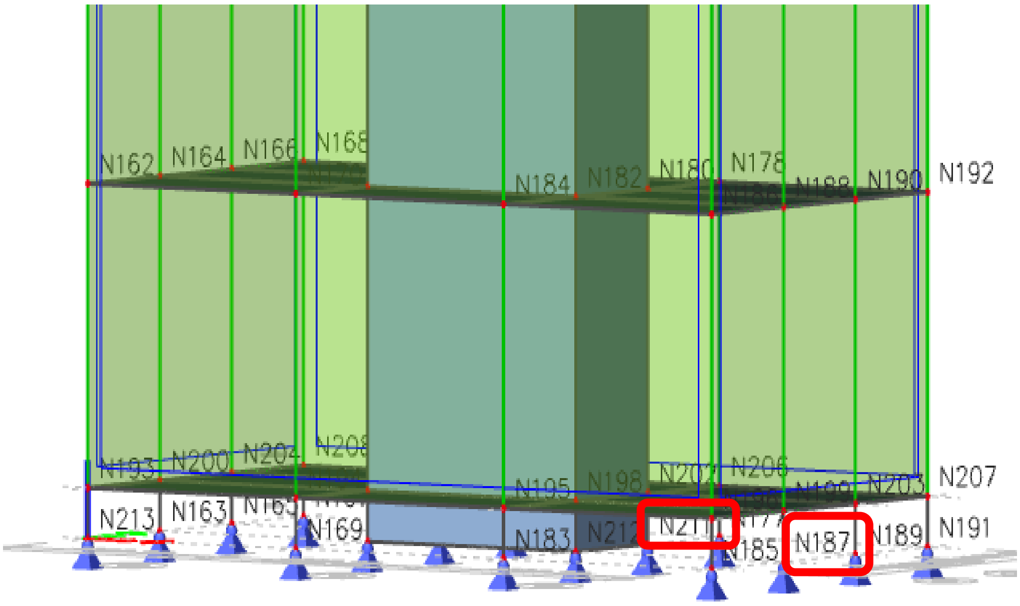

Figure 4.6: Nodes selected for side column and shear wall on single building.

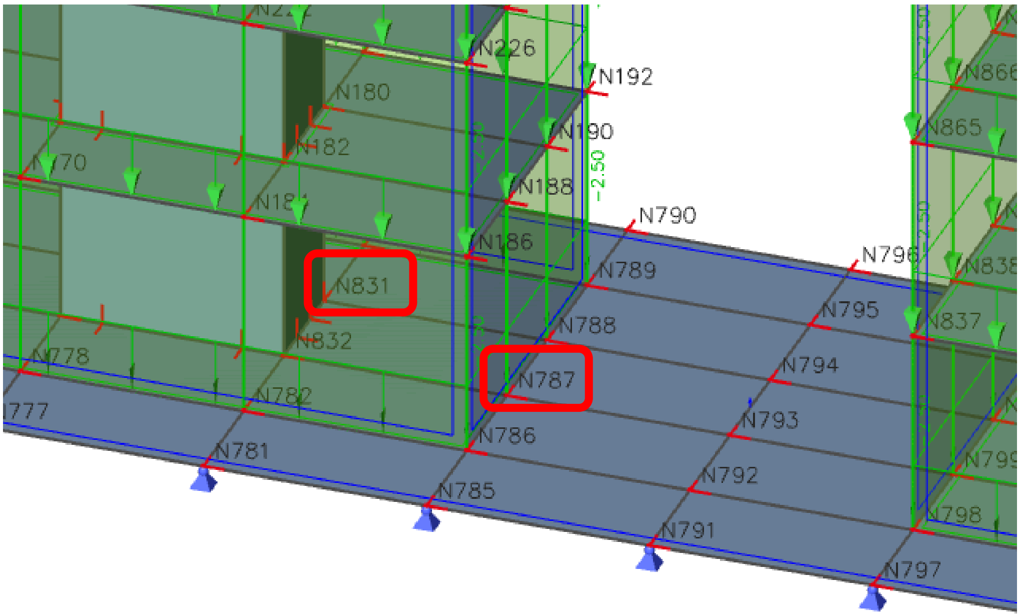

Figure 4.7: Nodes selected for side column and shear wall on twin building with podium.

Table 4.13: Percentage difference of reactions due to wind load with 90°, +CPE & +CPI.

Single Building	Podium Building	Structure	Rz (kN)		Difference (kN)	Difference (%)
			Single Building	Podium Building		
N187	N787	COL	-91.83	-101.73	9.90	11
N211	N831	SW	287.83	327.31	39.48	14

As tabulated in Table 4.13, the percentage difference of 11% and 14% were determined on the side column and shear wall of building models in the case for wind action with a wind direction of 90°, positive external and internal pressure. The column and shear wall in twin building with podium experience higher reactions compared to the single building. Several factors can contribute to the existence of a small difference in support reactions between the single building and the twin building with podium in terms of natural structural behaviour and the corresponding effect of the structure under wind action.

Principally, the structural behaviour of building is considerably affected by the podium structure, which has a larger plan dimension as in the twin building modelled in this study. The podium structure of the second model acts as a significantly large stiff box, which creates resistance to external action like lateral wind force acting on the structure above podium. Generally, a particular effect due to the podium is determined to enhance the lateral stiffness of the building structure, as shown in Figure 4.8. The occurrence of the effect at the interface level of building and podium is termed as a backstay effect. The backstay effect is established when a set of lateral forces is induced within a podium structure at the diaphragm and basement wall to equilibrate lateral load and moment incurred on building structure above the podium. The backstay effect has a significant impact on the shedding of the lateral load from the lateral load resisting system. Therefore, when wind action is incurred on the building structure, the generation of reaction on the podium floor is issued to overturn the shear force acting on the lateral structure of the building model. As a result, other reactions which commit to the percentage differences are incurred on columns and shear walls in diminishing the overturning effect of high-rise building due to base moment and shear under wind action.

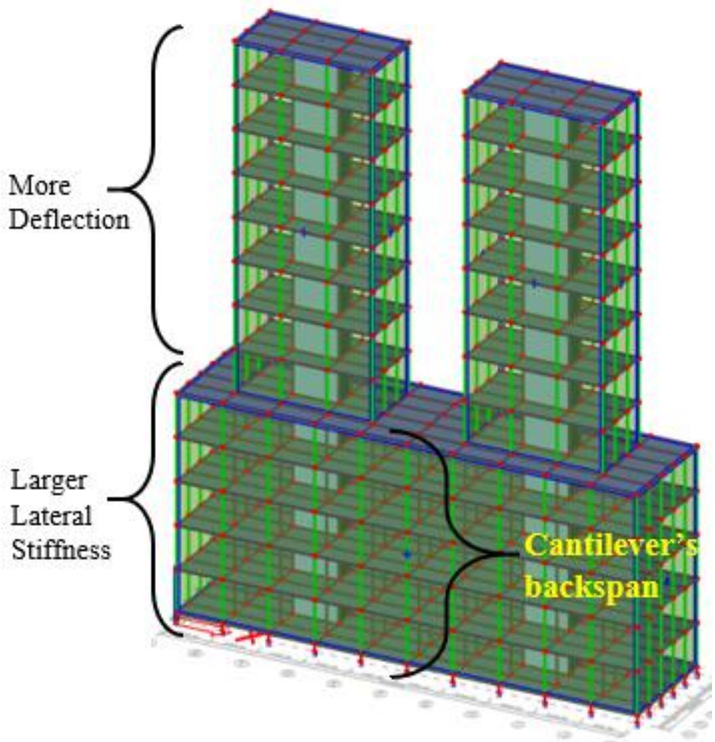

Figure 4.8: Backstay effect of twin building model with podium.

Besides, the interface level or podium diaphragm with certain stiffness and integrity also leads to the introduction of diaphragm action. The diaphragm action can be applied appropriately in interpreting the increase of reactions on shear wall for the second model. The interface level acts as the medium in transferring the horizontal load to the shear wall, which is the only lateral load resisting system for the twin building model with podium as illustrated in Figure 4.9. When wind load exerted on twin building, the restraining effect of the podium also arises due to the presence of strut and tie action on the interface level. The restraining effect of diaphragm results in the generation of reactions on podium diaphragm. The reactions generated are relatively transferred to the shear wall, which is structurally reliable for lateral load distribution. Accordingly, the load distribution from the podium to the shear wall comparatively contributes to the percentage difference obtained through data analysis.

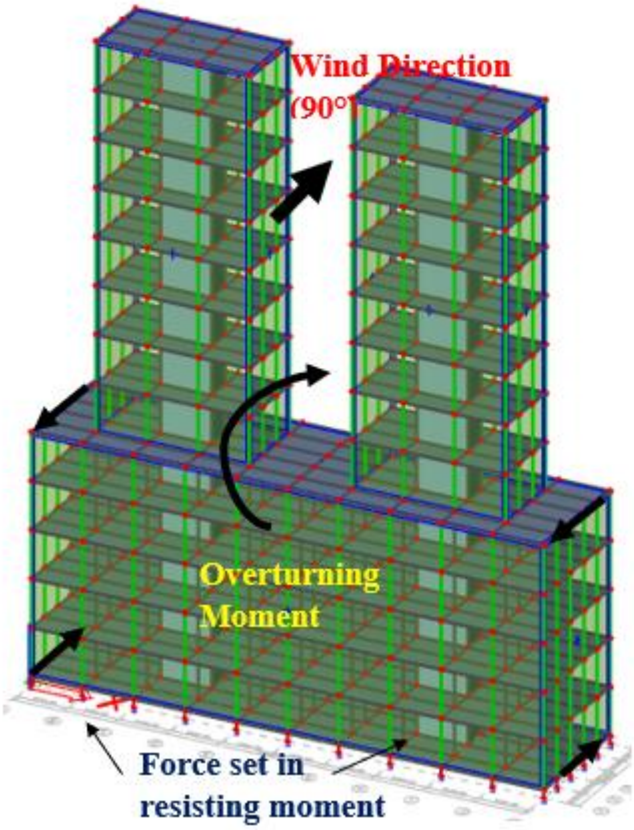

Figure 4.9: Diaphragm action of twin building model with podium.

Wind flow patterns and wind characteristics alter following the distributions of buildings surrounding the primary building model studied. Nevertheless, the presence of interaction and obstruction effect, which results in considerable alteration of wind pressure on building surface, cannot be overlooked. In this case, with a wind direction of 90°, the shielding effect is neglected as the two buildings of the second model do not upwind each and another due to the perpendicular arrangement of the two buildings to the wind flow direction.

Furthermore, due to the side-by-side configuration of the twin building model with podium, the phenomenon of channelling effect, which is another type of interaction effect of buildings, is critically reviewed. The channelling effect or venturi effect tends to affect the pressure coefficient on the side and back surface of buildings when a narrow passage appears between the twin building. Channelling effect of wind flow impacts on wind mean flow velocity as wind flow is accelerated because of the existence of pressure difference. For the second building model examined, magnification of channelling effect is developed as the building has larger crosswind width in reaching high slenderness ratio of building. The wind load acting on building structure is magnified as higher pressure is exerted on the outer side surfaces of the neighbouring buildings in which the crosswind effect is incurred. As a result, larger wind load leads to higher overturning moment acting in the crosswind direction towards both buildings. Thus, large reactions on side columns nearer to the neighbouring building are developed for the twin building model with podium. Besides, the overturning moment due to wind load also causes extra strut and tie action on podium diaphragm, which then leads to redundant reactions sustained by the shear walls of the second building model.

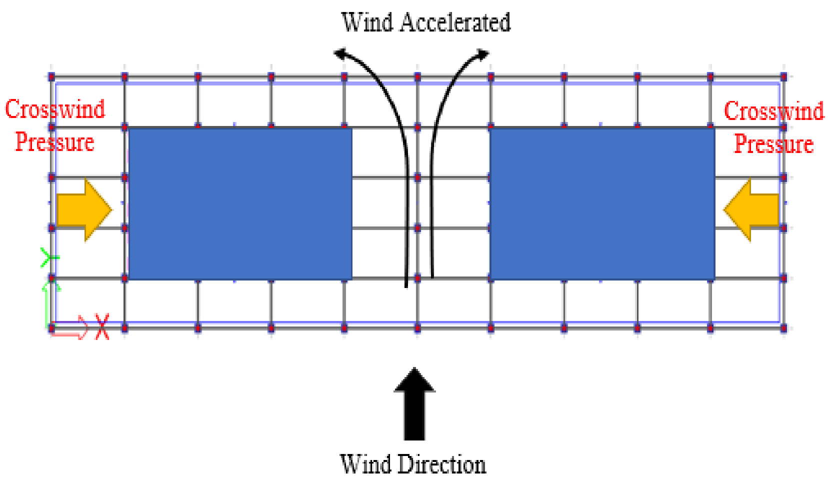

Figure 4.10: Channelling effect on twin building model with podium with plan view.

Lastly, the podium interface level, similar to the roof, experiences uplift of wind pressure during wind flow. Figure 4.11 shows the independent podium structure from the second building model taken for wind load analysis. Wind load causes a suction effect on the interface level, which tends to negative reaction force (downward) on the connection joints of columns and shear walls. As illustrated in Figure 4.12 below, the negative reaction of the side column (N787) and the shear wall (N831) are indicated. Instead of the contribution of positive reaction force on the side column and shear wall, this podium effect leads to the reduction of reaction values of vertical structure components. Hence, it is determined that the reaction force due to the uplift of the podium interface level contributing a trivial influence on the percentage difference obtained through the linear analysis of building models. The minor effect of podium reaction due to wind action can be neglected. Besides, it is determined that other factors, as discussed above, contribute significantly to the increase of reactions on the side column and shear wall examined.

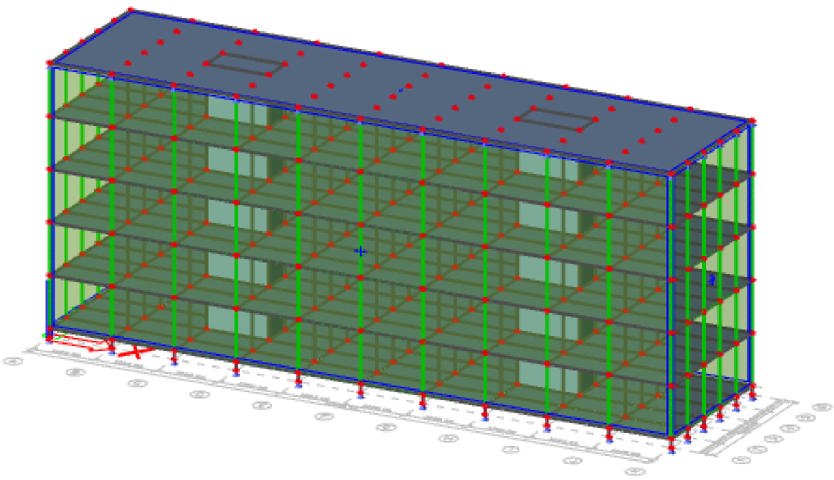

Figure 4.11: Individual podium structure detached from second model.

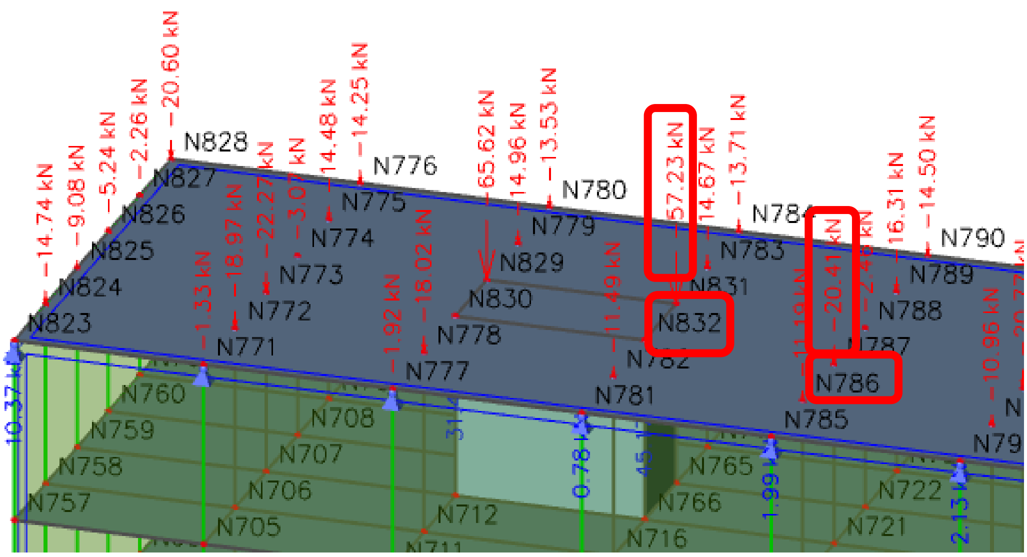

Figure 4.12: Reaction on podium interface level under wind load case of 90°, +CPE & +CPI.

4.5 Summary

In a nutshell, the evaluation of the comparative study between European Standard, British Standard and Australia & New Zealand Standard are performed and appropriate standard codes for wind load design is determined. The results of support reactions of both building models under the ultimate limit state and wind load for four types of combinations of wind characteristics are obtained through linear analysis. The differences in load and percentage difference due to wind effect are determined through calculation. Several factors contribute to the percentage difference between two building models are identified and discussed clearly with the illustration of figures.

CHAPTER 5

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

In this study, parametric study based on European Standard, British Standard and Australia & New Zealand Standard was implemented. Appropriate wind code was chosen and applied in software analysis. Two building models were constructed with SCIA Engineer. Linear analysis was performed on the building models to obtain the support reactions on column and shear wall. Support reactions between two building models were compared and percentage differences were identified. The factors contribute to the percentage differences were identified and depicted. The objectives of the thesis were achieved. Firstly, the parameters used in the wind load calculation according to the three wind codes were evaluated. Secondly, the effects of different wind loading evaluation based on the three wind codes were investigated. Thirdly, the building response under the ultimate condition for member forces in the shear wall and support reaction were compared. According to the objectives, findings of this study are determined and concluded as:

i. European Standard (EN 1991-1-4) is chosen for this study due to its larger wind effect on building structure than British Standard and Australia & New Zealand Standard. Larger base shear and base moment are obtained according to European Standard, which contributes to the higher safety design of high-rise building.
ii. European Standard considers structural factor which takes into account the factor of background response and resonance response.
iii. European Standard considers the distribution of wind pressure by parts on the vertical wall of the building. Different allocation of wind pressure on the sidewall and roof of the building is also executed according to European Standard.
iv. Under ultimate limit state with the consideration permanent, live and wind load, the percentage difference between 14% to 57% is obtained by comparing the single building model and twin building with podium.

Under the wind load, the percentage difference of between 0% to 20% is obtained.

v. The factors that contribute to the percentage difference are lateral stiffness and backstay effect of podium structure, diaphragm action of podium structure, channelling effect and individual podium reaction.

5.2 Recommendations for future study

Based on the current study conducted, recommendations are identified while considering the limitations determined through literature review, results and discussion executed in this project. It is recommended that:

i. Wind tunnel test could be carried out on scaled single building model and twin building model with podium to accurately identify the distribution of pressure coefficients on building surface which benefits the understanding of building response due to wind effect.

ii. CFD (computational fluid dynamics) simulation could be executed to monitor the wind flow around the building models. Wind interaction effect on building structure could be determined and explained by studying the wind flow features obtained though its advanced numerical techniques and rapid computing progress.

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APPENDICES

APPENDIX A: Loads Assignation on Single Building Model

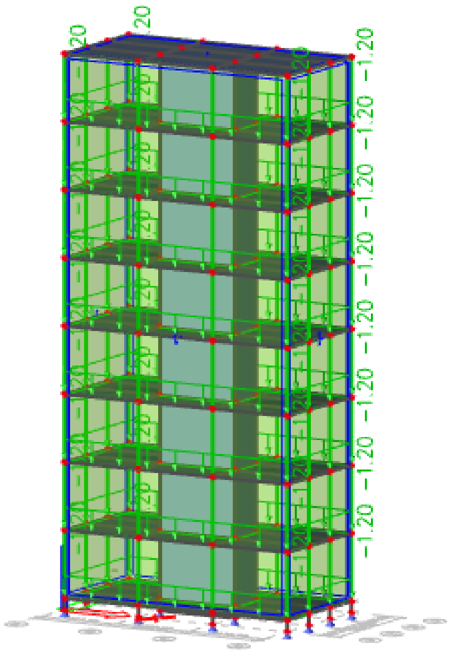

Figure A-1: Distribution of Finishing Load on Single Building Model.

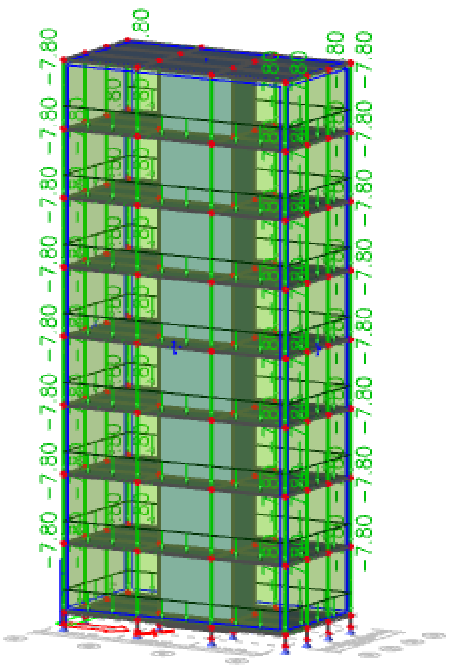

Figure A-2: Distribution of Brickwall Load on Single Building Model.

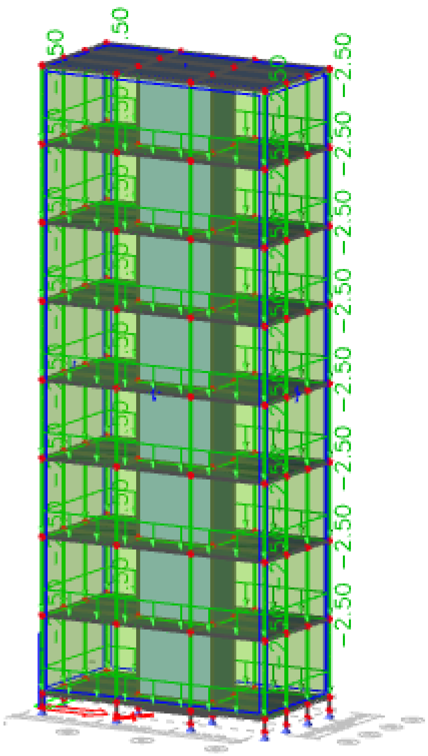

Figure A-3: Distribution of Live Load on Single Building Model.

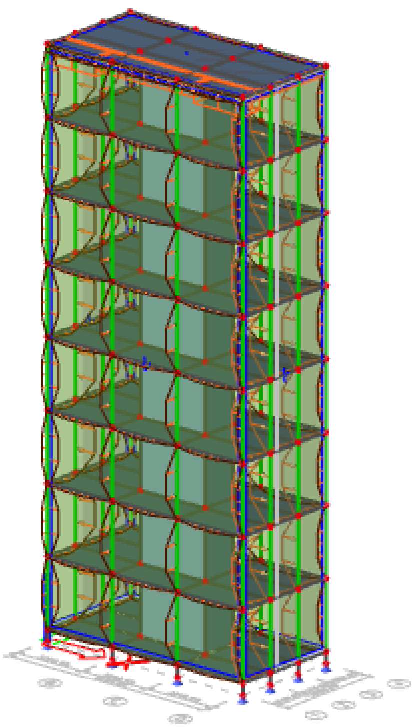

Figure A-4: Distribution of Wind Action (3DWind with 90°, +CPE &+CPI) on Single Building Model.

APPENDIX B: Loads Assignation on Twin Building Model with Podium

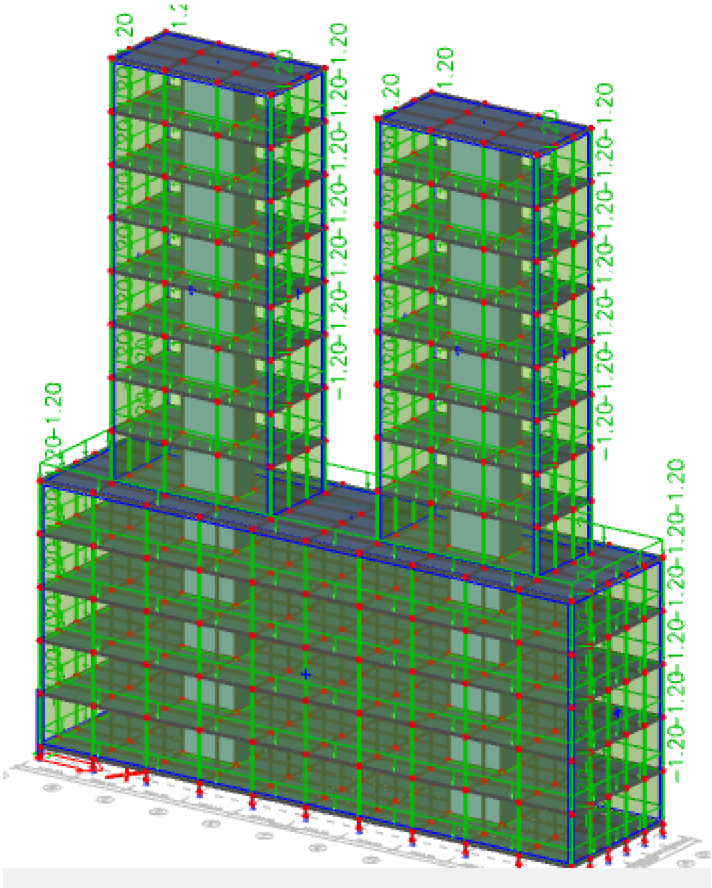

Figure B-1: Distribution of Finishing Load on Twin Building Model with Podium.

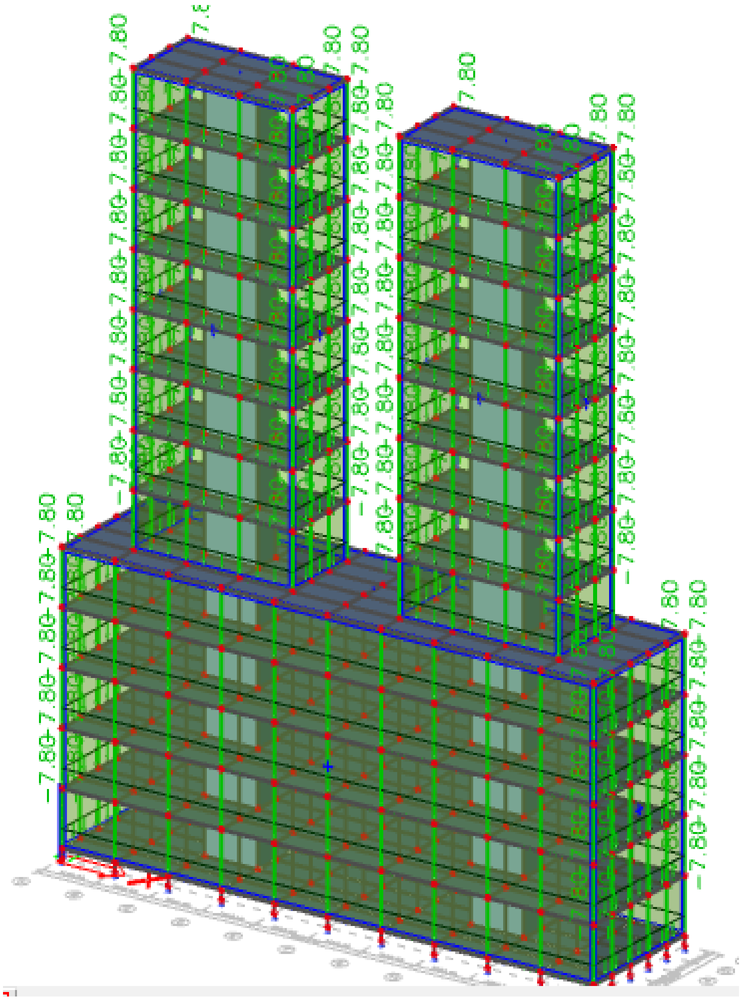

Figure B-2: Distribution of Brickwall Load on Twin Building Model with Podium.

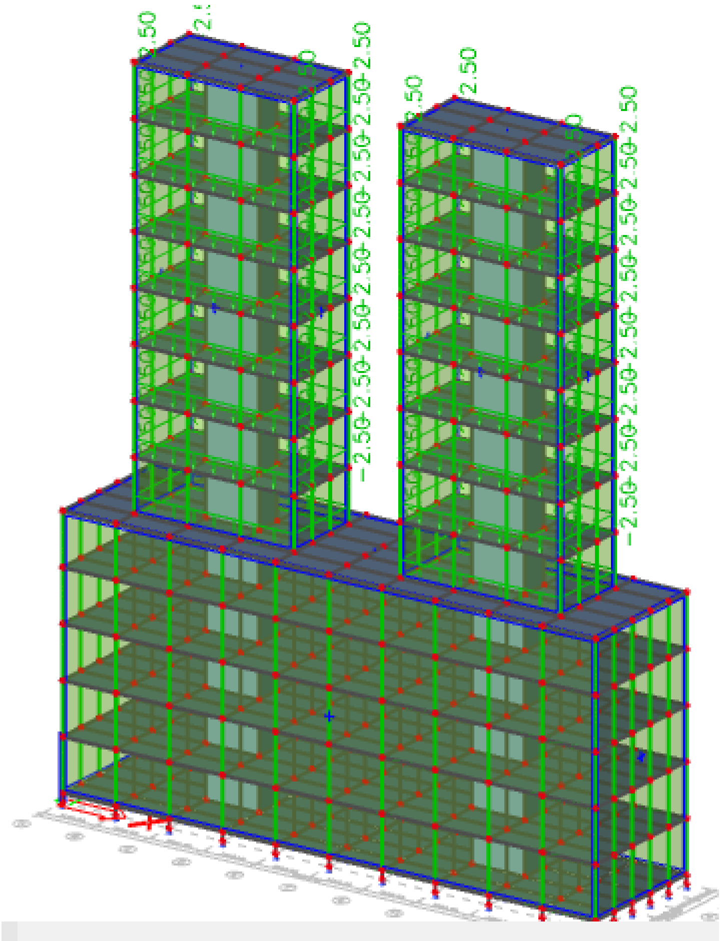

Figure B-3: Distribution of Live Load on Twin Building Model with Podium.

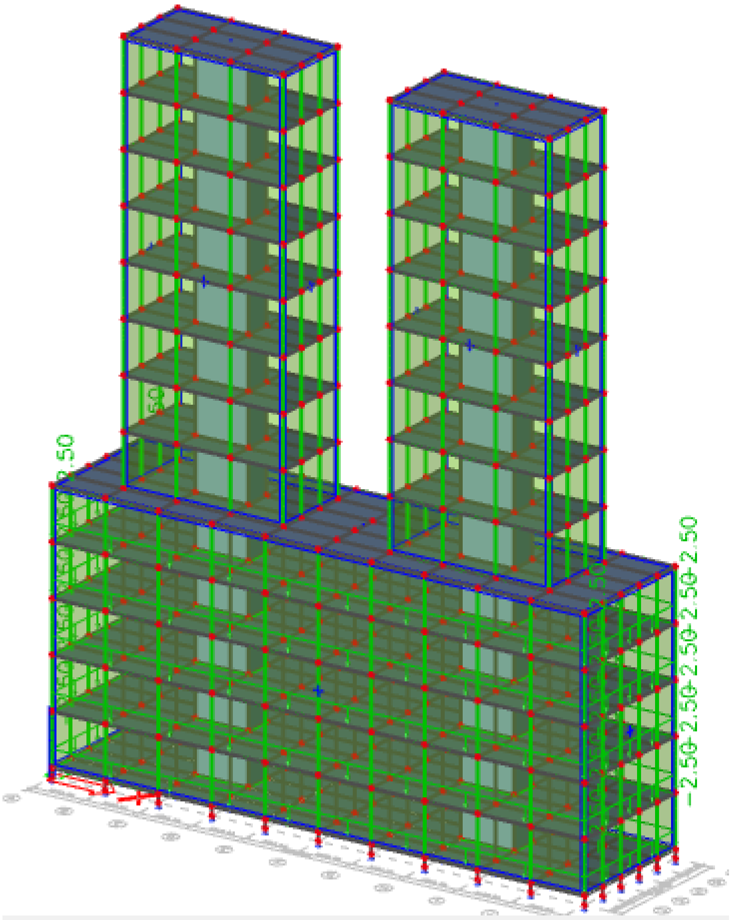

Figure B-4: Distribution of Traffic Parking Load on Twin Building Model with Podium.

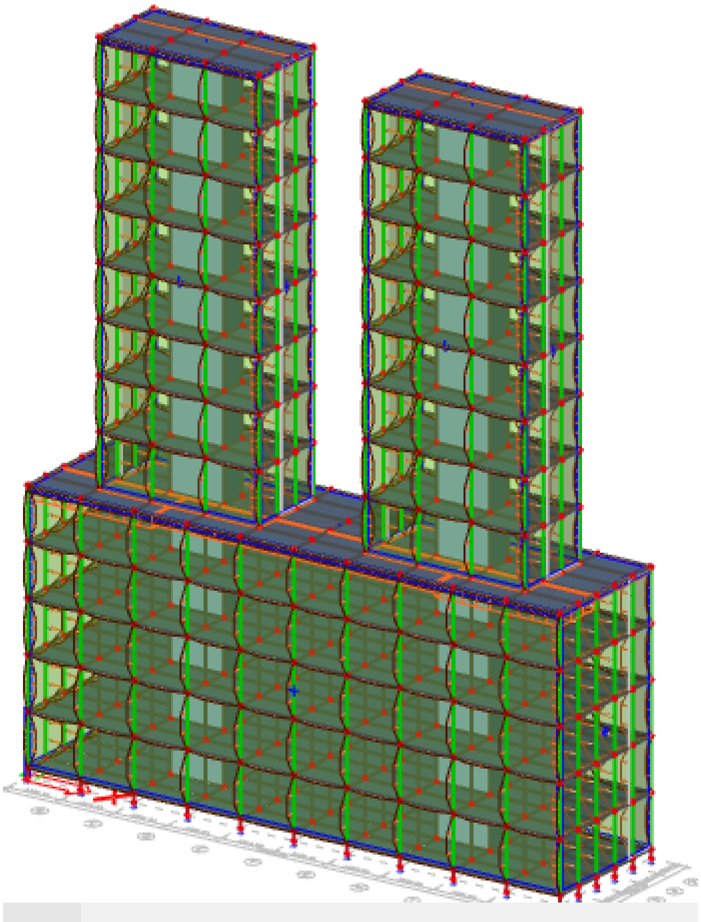

Figure B-4: Distribution of Wind Action (3DWind with 90°, +CPE &+CPI) on Twin Building Model with Podium.